Aalto-yliopisto
Insinööritieteiden
korkeakoulu

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Safety margin in serviceability limit state according to the 2019 draft of Eurocode 7: Applicability to embankments on fine-grained soils

Diplomityö, joka on jätetty opinnäytteenä tarkastettavaksi diplomi-insinöörin tutkintoa varten.

Espoossa 25.05.2020
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Abstract of master's thesis

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Master programme Geoengineering **Code** ENG23

Thesis supervisor Associate Professor Leena Korkiala-Tanttu

Thesis advisor(s) MSc. Monica Löfman, MSc. Panu Tolla

Date 25.05.2020 **Number of pages** 66+26 **Language** English

Abstract

Determination of characteristic values of soil properties is an essential step in the verification of serviceability limit states such as total settlements of embankments founded on fine-grained soils. These values provide a safety margin that accounts for sources of uncertainty such as inherent spatial variability of the properties within the soil mass and sampling uncertainty related to the extent of the investigations. However, modification of soil parameters results in deviations from the actual stress-strain behavior of soils that might be significant for certain soil conditions and characteristics. For instance, the nonlinear strains exhibited by soft Finnish clays in oedometer tests that are accounted for by the tangent modulus method.

The aim of this thesis was to quantify the safety margin obtained from characteristic values in settlement calculations. Four cases from Finland with different soil conditions were analyzed. Several total settlements analyses were carried out to compare values of settlements calculated from characteristic values with values from the mean of data sets of derived soil properties (i.e., best estimates). The analyses were made as functions of the range of coefficient of variation reported in the literature for the relevant soil parameters used in the calculations. Two different deformation models were used to calculate the settlements: tangent modulus method and compression index method. The European Committee for Standardization (CEN) is planning to publish a revised version of Eurocode 7 where a statistically based equation for the determination of characteristic values will be included. This equation was used for the determination of characteristic values in this study. Likewise, an alternative equation was also used for one case in which, a purely statistical approach cannot be applied.

The results show that large conservatism is induced by the strong nonlinearities of the stress-strain behavior of some soft Finnish clays when characteristic values of soil properties are used in settlement predictions. The results are especially critical for the upper bound of the range of the coefficient of variation selected. The characteristic values fail to provide a harmonized safety margin for all the cases because of the different stress-strain behavior features. Therefore, characteristic values of soil parameters used in the tangent modulus method have to be estimated cautiously to avoid unnecessary safety margin. Likewise, the equation from the revised version of Eurocode 7 should be avoided in cases where limited samples are available because settlements are considerably overestimated.

Keywords Serviceability limit state, Eurocode, characteristic values, settlements, embankments, fine-grained soils.

Acknowledgments

This thesis has been done at Aalto University in the Mineral-based materials and mechanics research group. The study was a part of a project funded by the Finnish Transport Infrastructure Agency. (Väylä in Finnish).

I would like to thank my supervisor Leena and my advisors for all your time, help, kindness, patience, and valuable comments. Special thanks to Professor Leena for all the support and encouragement you provided me. Also, I would like to thank Panu Tolla and Veli-Matti Uotinen from Väyla for making this work possible and who helped me to considerably improve the quality of this document. Thank you for your observations and enjoyable meetings where I learned many practical things.

Many thanks to my advisor Monica Löfman. She is not just an amazing and talented researcher with high skills who can easily figure things out when you are struggling to see a solution. She is also one of the coolest, down to earth persons I have ever met. Thanks to Henry Gustavsson for making me part of this research group, offering me his help, friendship, and advice.

Finally, thanks to my amado husband for being a constant and unalterable support and source of motivation in my life. Many thanks my talented colleagues Silin, Jenni, Raisa, Francisco, Lasse, and Mikko.

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1 Introduction

In the last decades, uncertainty related to the inherent spatial variation of soil properties has received considerable attention as it affects the probabilities of exceeding certain conditions leading to failure (Sivakumar & Mukesh, 2004), or a loss of serviceability in geotechnical problems. These conditions are referred to as limit states, as they define the point where structures cease to fulfill performance requirements established for their design, such as tolerable displacements in footings (Zhang & Ng, 2005). In addition to natural variations in soil properties, other sources of uncertainty are present in geotechnical design, such as the ones related to field and laboratory investigations that include measurement errors, statistical uncertainty, and transformation errors (Phoon & Kulhaway, 1999a). Statistical uncertainty is relevant in the case of a limited number of samples, whereas transformation errors are the result of applying empirical models in cases where the design soil property has not been measured directly (Uzielli et al., 2006). To deal with uncertainties, geotechnical engineers have at their disposal several options for the selection of the design values of soil properties from the derived values obtained through field and laboratory tests. One of these options is the estimation of a characteristic value, defined as a cautious estimate of the real value of the soil property influencing a limit state, which has a certain probability of having a more unfavorable value. Once the characteristic value of a soil property has been estimated, the design value is calculated by applying a partial safety factor to the characteristic value. Partial safety factors are also applied to actions or the effect of actions, and/or resistances (Länsivaara & Poutanen, 2013).

European standard Eurocode 7 (CEN, 2004) provides guidelines on the estimation of the characteristic value of soil properties and the values of partial safety factors to be used under different design situations. According to Eurocode 7 (EC7), designers may select characteristic values based on their experience and assessment of the problem or based on statistical methods. Regardless of the method used to estimate the characteristic value, this value can correspond to a cautious estimate of the mean value or a cautious estimate of the upper or lower bound of the soil property (CEN, 2004). On the other hand, partial safety factors account for any unfavorable deviation of the soil property from its characteristic value and the uncertainties in the model used in calculations (Länsivaara & Poutanen, 2013). Thus, the characteristic value together with the partial factor provides a safety margin for the design values of soil properties to be used in the verification of limit states.

Another term related to the selection of values of soil properties in geotechnical problems is the best estimate value. The best estimate is not a term defined in the current version of EC7. However, a new version of the standard is under review, referred to as the October 2019 draft, where the best estimate is defined as the most probable value of the soil property. The most probable value of a data set following a normal distribution corresponds to the mean and median value, although other values below or above the mean can be selected as best estimates. As the most probable value, the best estimate is treated as the most accurate estimation of the real settlements during the design stage, which can be adjusted later on in back-analysis to match predictions on soil behavior with measured values in the field. This means that the best estimate as a value used for prognosis, might provide little or no safety margin and for that reason, the characteristic values serve as cautious estimates of the mean value of a soil property and the values used in the limit states verification.

However, characteristic values might result in too conservative estimates of design values, especially if the best estimate already carries some conservatism. Likewise, there is considerable subjectivity in the selection of the best estimate and in general, in characteristic values based on non-statistical methods. In fact, designers can select very different values of characteristic values according to their experiences and their evaluation of the design situation (Orr, 2017). The lack of a consistent design value might lead to unreliable designs. For instance, in geotechnical problems such as embankment design, working with unreliable values of design soil properties might lead to an underestimation or an overestimation of soil settlements. In cases where the embankment is founded on fine-grained soils, settlements are evaluated against allowable values, which constitutes the serviceability limit state (SLS) verification established by EC7. The verification of SLS requires design values that are obtained by applying partial factors to characteristic values or best estimates. The partial factor for SLS defined by EC7 is 1.0. Thus, there is not any safety margin beyond the estimation of characteristic values of soil properties in embankment design. Additionally, if the selection of characteristic values fails to yield sufficient safety margin for settlement calculations, partial factors higher than unity might be needed. On the other hand, characteristic values might result in an unnecessarily large safety margin as explained before. Actually, the current version of EC7 states that other partial factor than the unity can be used for SLS verifications if the National Annex provides one. However, currently in Finland, a partial safety factor of 1.0 is the only accepted value in SLS. This issue could be addressed by quantifying first, the safety margin provided by characteristic values in total settlement calculations for assessment of a possible future calibration of partial factors in SLS. For instance, using data from real cases of embankments founded on fine-grained deposits in Finland, and comparing for each case, the different settlements values obtained from using best estimate and characteristic value approaches separately. If settlement calculations turn out to be sensitive to the use of characteristic values estimated through statistical methods when compared to the selection of the best estimate, recommendations could be given for future calibration of a partial safety factor. A partial factor can ensure a safety margin high enough for compensation of any uncertainty underestimation involved in the selection of design values of soil properties.

Therefore, the aim of this thesis is to evaluate the influence of using best estimate and characteristic values of soil parameters on the estimated total settlements and their corresponding safety margins. In order to achieve this goal, this study uses data from four real cases to calculate settlements by using best estimates and characteristic values of soil properties. All the selected cases correspond to test embankments founded on fine-grained soil deposits located in Finland, where soil properties were obtained by means of laboratory testing. The best estimate values of these soil properties are treated as unmodified parameters and characteristic values as modified parameters. Then, the settlements resulting from unmodified and modified values are compared by obtaining the ratio of the settlements from characteristic values to values from best estimates. This ratio is an indication of the safety margin resulting from using the characteristic values of settlement parameters.

This thesis focuses only on the values of settlements in embankments built on fine-grained soils such as clays and silts. These values of settlements correspond to the ones used for

SLS verification. Likewise, only total settlement from primary consolidation is calculated, except for one of the cases, where a time-settlement analysis is performed with unmodified and modified parameters to evaluate the sensitivity of the results to the use of characteristic values. The estimates of the time-settlement analysis correspond to the degree of primary consolidation achieved after 2 and 3.5 years. Therefore, settlements resulting from secondary consolidation are excluded from the scope.

The rest of this thesis is divided into six chapters. Chapter 2 explains the different models and the experimental procedure carried out for the determination of the different parameters needed in settlement calculations. Chapter 3 reviews the European standard Eurocode 7 and other literature on the tools and concepts used during the thesis. Chapter 4 describes the study cases used to achieve the aim. Chapter 5 outlines the methodology used in this thesis. Chapter 6 presents the results of the different settlement calculations performed in each case and discusses how the selection of modified and unmodified parameters affects the calculated settlement. Chapter 7 concludes the thesis by discussing the reliability of using best estimates and other non-statistically based design values for settlement calculations and gives recommendations about the possible use of partial factors different from the unity.

2 Theoretical framework

Calculation of soil settlements in fine-grained soils involves the application of a deformation model capable of providing relatively accurate estimates of the strains developed under specific stress states. This chapter presents the different models and theories that are commonly used for evaluation of the consolidation of fine-grained soils, along with a description of the experimental procedure carried out for the determination of the different parameters needed in settlement calculations. In the case of Finnish soft clays, a special deformation model is needed to account for the nonlinearities of the characteristic compressibility curve of these soils. This model, known as Janbu´s model, is also introduced in this chapter, as it is the main model used in Finland and the present study.

2.1 Soil compressibility

Compressibility is the property of soils defining the process of compression or consolidation of the soil in undrained and drained deformation state. Under an increase in total vertical stress, soils have the capability of experiencing a volume decrease at a rate that depends mainly on the permeability of the soil (Smith, 2014, p. 355, Craig, 2004, p.75). In fully saturated soils with low permeability properties such as clays and silts (cohesive, fine-grained soils), a rapid load increment will not give any time for the water to escape from the voids (Ishibashi & Hazarika, 2015, p.166). This immediately generates a pore pressure increase without changes in effective stress and no consequent volume change (Smith, 2014, p.316). This is known as initial settlement, where the increase in total vertical stress is transferred entirely to pore water, and the soil is considered to be in undrained condition (Craig, 2004, p.74). Eventually, water starts draining from the soil in a process known as pore pressure dissipation. The increment in total stress is transferred to the soil matrix, increasing the interparticle forces with a consequent consolidation of the soil in a new deformation state known as drained state (Craig, 2004, pp.74-75). At this point, the primary consolidation settlement is taking place. Total consolidation is attained once the pore pressure dissipation is complete (Ishibashi & Hazarika, 2015, p.166). Consolidation is a slow process in soils with very low permeability, whereas high permeability soils have immediate settlements due to rapid water drainage (Craig, 2004, p. 75). Unsaturated soils, which are soils partially saturated with water and partially saturated with air, will experience immediate compaction after being loaded due to the exclusion of air from the voids and the instant change in effective stress, with a change of the pore-water pressure.

Settlements are assumed to consist of three types: initial settlement (S_i), primary consolidation settlement (S_c), and secondary settlement (S_s). The final settlement of the soil is called total settlement (S_t), and its magnitude is the sum of the above-mentioned types of settlement ($S_t = S_i + S_c + S_s$) (Ishibashi & Hazarika 2015, p.163).

2.1.1 Initial settlement

In initial settlement, the incremental load is first transferred to water in the voids generating an excess in pore water pressure Δu, and therefore Δu will be equal to the total stress increment $\Delta\sigma$ (Smith, 2014, p. 355). Vertical deformation during initial settlement consists of a change in shape; with no volume change involved (Smith, 2014, p. 316).

The initial settlement is assumed to be zero if lateral strain is also zero (Craig, 2004, p. 238). The assumption of zero lateral strain is a reasonable approximation for cases where the loaded area is too wide with respect to the thickness of the compressible layer. However, in some cases a significant amount of lateral strain will take place (Craig, 2004, p.237). Initial settlement is expected to occur during or right after construction (Ishibashi & Hazarika, 2015, p.166).

Initial settlements are an instantaneous elastic response of the soil to an increment in vertical stress (Ishibashi & Hazarika, 2015, p.166). Therefore, the amount of initial settlement can be estimated using the elastic theory (Craig, 2004, p. 155). This involves the estimation of elastic constants such as Poisson's ratio (v) and the modulus of elasticity (E) of the soil. In the case of homogeneous clays, it is normally assumed that E remains constant with depth (Craig, 2004, p.156). The estimation of E can be done by means of laboratory testing. For instance, it can be obtained from the consolidated undrained triaxial test (Smith, 2014, p.320). Nevertheless, the amount of initial settlement in clays is usually considerably small such that, it cannot be separated from primary consolidation analyses.

2.1.2 Primary consolidation

Primary consolidation or consolidation settlement is a time-dependent process mainly attributable to fine-grained soils that starts with the pore pressure dissipation in fully saturated conditions induced by a load increment in the soil. After the initial settlement has ceased and water starts dissipating, the volume of the soil unit starts decreasing causing soil settlement (Ishibashi & Hazarika, 2015, p.192). The stress state during the consolidation process is characterized by an increment in the effective vertical stress and water pressure, both produced by the additional load on the ground. For instance, an embankment built on fully saturated, fine-grained soil represents a load increment that will produce in the same amount an increase in the total vertical stress ($\Delta\sigma$), that is in turn, the sum of the additional pore pressure (Δu) and the additional effective stress ($\Delta\sigma'_o$), as shown in Equation (1). The increment in effective stress ($\Delta\sigma'_o$) is a result of the load increment being transferred to the soil grains, producing intergranular pressure and consequently, compression (Smith, 2014, p. 358).

$$\Delta\sigma = \Delta\sigma'_o + \Delta u \tag{1}$$

Once the excess pore pressure reaches a negligible value, it is assumed that total primary consolidation has been attained (Smith 2014, p. 361). At this point, the applied total pressure equals the effective vertical stress in the specimen $\Delta\sigma_v = \Delta\sigma_v$' (Smith 2014 p. 356).

The settlements induced by a wide embankment with respect to the thickness of the compressible ground on which it is founded constitutes a one-dimensional consolidation problem. In this case, the soil that is adjacent to the loaded soil unit restricts the lateral deformation of the soil (Terzaghi et al., 1996). This means then, that excess pore water can be only drained upwards and downwards. The presence of an impermeable soil layer underneath the compressible soil also favors the one-dimensional consolidation condition (Terzaghi et al., 1996). In this case, pore water can be drained in only one direction, which is upwards (Smith, 2014, p. 357).

One-dimensional consolidation problem can be approached by Terzaghi's classical one-dimensional consolidation theory (Terzaghi, 1925), where consolidation settlement is a time-dependent process controlled by the pore water pressure gradient. An alternative to Terzaghi's theory is Janbu's consolidation theory (Janbu, 1970), founded on the same pre-assumptions of Terzaghi's theory but stating that primary consolidation is controlled by the dissipation of a rest strain. Nevertheless, in both theories a differential equation is derived, which allows calculating the time dependency of settlements.

2.1.3 Secondary consolidation

Secondary settlement or creep is the reduction in the volume of a soil mass due to the application of a sustained load and characterized by a constant effective stress phase as most of the load has been transferred from the pore water to the soil grains. Creep occurs in all kinds of solid materials producing a slow and permanent deformation under persisting loading. In the case of soils, creep or secondary settlement is a very slow and continuous settlement that happens in all kinds of soils, especially in the most compressible ones. In the classical one-dimensional theories, it is assumed that primary and secondary consolidations happen in two separate stages. However, creep effects also occur during the primary consolidation phase and they can be included in one-dimensional time settlement calculations (Länsivaara, 1999). Likewise, during secondary compression there is also excess pore water pressure generating water discharging, but it is too small to be included in the secondary consolidation analysis (Terzaghi et al., 1996).

2.2 Analysis of consolidation - Terzaghi´s one-dimensional consolidation theory

Calculation of primary deformations was not possible until 1923 when Terzaghi published his one-dimensional consolidation theory (Leonards & Ramiah, 1960). The pore pressure-based consolidation theory is governed by a partial differential equation describing the dissipation of excess pore water pressure (u) within a consolidating soil over time (t) and relative to space domain (z). Terzaghi's equation can be written as (Terzaghi & Peck, 1961, p. 248):

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \tag{2}$$

Where u is excess pore pressure [kPa]
t is time [s]
z is depth [m]
C_v is the coefficient of consolidation [m^2/s].

The velocity at which the soil settles depends on C_v, which is defined in Equation 3 (Terzaghi & Peck, 1961, p. 248):

$$C_v = \frac{kM}{\gamma_w} \tag{3}$$

Where k is permeability [m/s]

M = compression modulus [kPa]
γ_w = unit weight of water [kN/m^3].

The application of Terzaghi's one-dimensional consolidation equation must satisfy the following assumptions (Smith, 2014, p. 356; Leroueil et al., 1990, p. 93: Craig, 2004, pp. 245-246):

- The seepage flow and consolidation of the soil occur in the same direction (vertical).
- Darcy's law is valid.
- The coefficient of permeability and coefficient of volume compressibility is constant.
- Soil is homogeneous and fully saturated.
- Volumetric strains caused by changes in effective stress, only take place in the voids, as water is incompressible.
- Strains are small

A solution of Terzaghi's equation using Fourier's series requires calculating the time factor T_v (Terzaghi & Peck, 1961, p. 249).

$$T_v = \frac{C_v}{h^2} t \tag{4}$$

Where *t* is consolidation time, which can be written as (Equation 5):

$$t_u = T_v \frac{h^2}{C_v} \tag{5}$$

The solution of equation 2 allows determining excess pore pressure at a time t for any point within the soil. The mathematical solution has the following form (Equation 6) (Ishibashi & Hazarika 2015, p.192):

$$u(z,t) = f\left(\Delta\sigma; \frac{z}{H}; T_v\right) \tag{6}$$

Where H is the thickness of the consolidation layer [m]
$\Delta\sigma$ is the stress increment [kPa].

A graphical solution of Terzaghi's equation is shown in Figure 1, where the degree of consolidation U_z (Equation 7) can be obtained for a specific time factor T_v. The degree of consolidation can be written as follows (Terzaghi & Peck, 1961, p. 81):

$$U_z = \frac{\delta(t)}{\delta_p} \tag{7}$$

Where U_z is the degree of consolidation for primary settlement at time t
$\delta(t)$ is the primary settlement of the structure at time t
δ_p is the final primary settlement of the structure at time t.

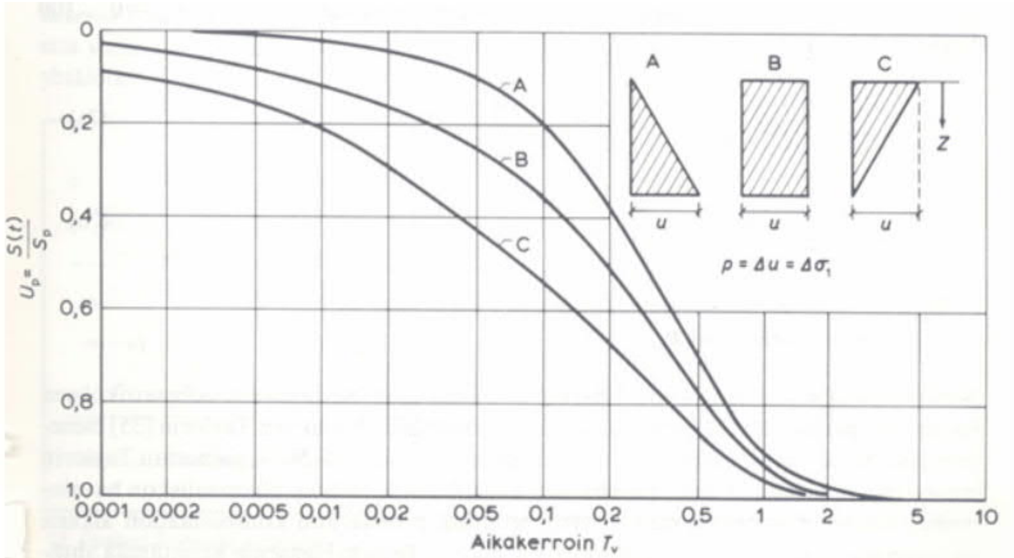


Figure 1. Time factor T_v as a function of consolidation degree. The three polygrams and curves correspond to three different water dissipation patterns according to Terzaghi's theory (Korhonen 1985, p. 289).

2.3 Oedometer test and compressibility parameters

The design of wide embankments founded on compressible soil requires the computation of settlements. When settlements exceed an acceptable value, the ground-supported structure has to be redesigned (Terzaghi et al., 1996). This constitutes the design criterion of the serviceability limit state verification that will be later discussed in chapter 3. In order to compute these settlements, a compression test is carried out to obtain the necessary consolidation parameters based on the stress-strain relationship of the soil. The test is performed on laterally confined specimens to simulate the one-dimensional conditions (zero lateral strain) under which, settlements of saturated clays and silts take place (Craig, 2004, pp. 227,237). In an oedometer test, a soil sample is placed in a cylindrical confining ring that prevents any lateral deformation during the application of load increments. Full saturation of the sample is secured by covering the sample with water. On top and beneath the sample, a porous stone is placed to facilitate the drainage in the vertical direction. In some versions of the test, drainage is allowed only from the top, and the pore pressure is measured at the bottom (Terzaghi et al. 1996).

During the test, the sample is subjected to compressive stress by the application of an incremental vertical load by a certain number of stages, and the resulting settlements are measured at each stage. Each load step is generally kept constant for a period of 24 h, during which, compression readings are taken at certain intervals (Craig, 2004, p.228). At the end of each loading step, the excess pore pressure has been completely dissipated ($\Delta u = 0$), and the end of primary consolidation (EOP) has been reached. This allows obtaining the EOP void ratio e_p (Terzaghi et al., 1996). According to the selection of the loading condition, the test performed can be an incrementally loaded oedometer test (ILOT) or a constant rate of strain test (CRS). In a standard ILOT procedure, the incremental load is doubled on each step and maintained for 24 hours. For oedometer CRS tests, the sample is compressed at a constant displacement rate with varying load.

Compression steps are usually followed by unloading steps, during which, the soil increases in volume (i.e., swelling).

2.4 Oedometric curve: Compression and swelling index

Using the values of vertical deformation and consolidation pressure obtained in an oedometer test, a plot expressing the relationship between void ratio (e) and effective stress (σ'_v) is obtained. The EOP void ratio – effective stress relationship is independent of the length of the primary consolidation stage. Therefore, the oedometric curve can be used directly to estimate primary settlements of fine-grained soils in the field, by taking the change in void ratio (Δe_p) from the initial effective overburden stress (σ'_{vo}) to any final consolidation pressure (Mesri & Choi, 1985). It is common to use logarithmic scale to represent the pressures as shown in Figure 2, where different parts of an Oedometric curve corresponding to compression, unloading, and reloading can be appreciated. An approximately linear relationship between the void ratio and stress in the semi-logarithmic scale has been observed for most clays reported in the literature (Mesri & Choi, 1985). The slope of this straight part of the compression curve corresponds to the compression index (C_c) defined by Equation (8). C_c is the most widely used parameter to describe the settlement properties of fine-grained soils. Similarly, the slope of the straight part of the unloading curve is the swelling index (C_s) (Equation 9).

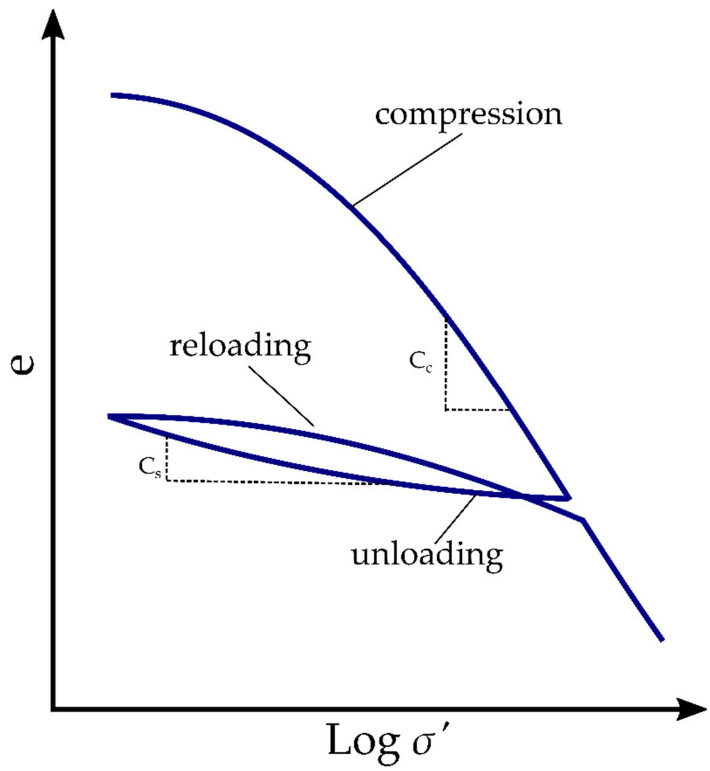


Figure 2. Oedometric curve with the compression, unloading, and reloading parts.

$$C_c = \frac{\Delta e}{\Delta(log_{10}\sigma)} \tag{8}$$

$$C_s = \frac{\Delta e}{\Delta(log_{10}\sigma)} \tag{9}$$

The shapes of the curve in Figure 2 are related to the history of the clay (Craig 2004, p.229). In the case of clays of Nordic Countries such as the soft Finnish clays of post-glacial origin, a linear relationship between the void ratio and stress in a semi-logarithmic scale cannot be applied (Ravaska & Vepsäläinen, 2001). Thus, the compression index *Cc* might not yield accurate values describing the settlement properties of these soils. This led to the development of a new deformation model that can apply to the Scandinavian fine-grained soils, the Ohde-Janbu model, (Janbu, 1970), which will be discussed in chapter 2.8.2.

2.5 Preconsolidation Pressure and state of consolidation

As it was observed in Figure 2, in the first part of the compression curve of the e-logσ'_v plot, the soil exhibits a "stiff" behavior. This part is known as the recompression range, which under field conditions starts from the in-situ void ratio (e_0) and the initial effective overburden pressure (σ'_{vo}), and ends at the preconsolidation pressure, after which, soils start "softening" their deformation response to loading (Mesri & Choi, 1985). The preconsolidation pressure (σ'_p), is the maximum stress state that the soil has been subjected to during its geological history (Craig, 2004, p.231). Soils have previously adjusted to this high stress level and therefore, no significant volume strains are developed in response to vertical stresses below σ'_p value. Thus, during the recompression range only minor interparticle slip takes place, which is mainly elastic deformations. Just above the preconsolidation pressure, plastic strains start developing because the soil interparticle bonding is being subjected to progressive destructuration (Terzaghi et al., 1996). This marks the start of the compression range with major structural changes that are reflected in high values of strain. In soft clays with a great interparticle bonding, the yielding point is well defined as the transition from recompression to compression is abrupt with a brittle bonding breakdown. (Mesri & Choi, 1985). Figure 3 shows the general features of a e-logσ'_v curve of soft clays.

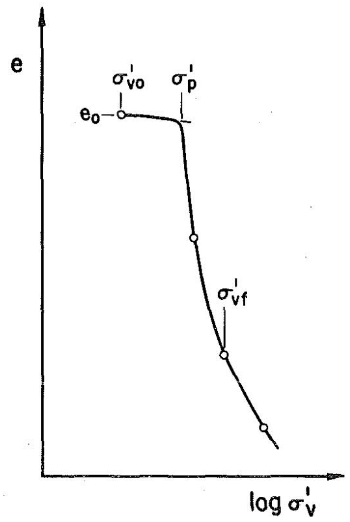


Figure 3. General form of a e-logσ'_v plot of soft clays. (Mesri & Choi, 1985).

The state of consolidation of natural soil in the field is defined by e_0, and σ'_{vo}. Likewise, the magnitude of the preconsolidation of a soil is expressed through the overconsolidation ratio (OCR), which is the ratio between σ'_p and σ'_{vo}. Soils with OCR>1 have been subjected to a previous consolidation under effective vertical stresses higher than the existing σ'_{vo}, and they are referred to as overconsolidated soils. Soft clays and silts with

OCR higher than the unity have experienced mechanisms of secondary compression in addition to geological loading and unloading, which also contributes to the high compression that these soils show after the recompression range (Terzaghi et al., 1996). On the other hand, soils with OCR=1, which are called normally consolidated and whose existing σ'_{vo} is the highest stress level experienced, have recently completed primary consolidation. Therefore, there is a lack of recompression behavior when subjected to an increment in vertical stress. For instance, after the construction of an embankment, the additional pressure and the settlements will be within the compression range. These soils are generally young soil deposits.

When the oedometer test is performed and the e-logσ'_v plot is obtained, the compression range corresponds to the overconsolidated area of the curve and beyond the preconsolidation pressure, the soil sample behaves as normally consolidated. Figure 4 shows an oedometric curve of an intact (blue) and a reconstituted material (red). This latter curve is called intrinsic compressibility due to the absence of bonding. The linear behavior in the compression range is an effect of bonding but after the yielding point (normally consolidated stage) the compression curve will gradually start converging with the intrinsic compressibility as a result of the destructuration of bonds (Mataić, 2016).

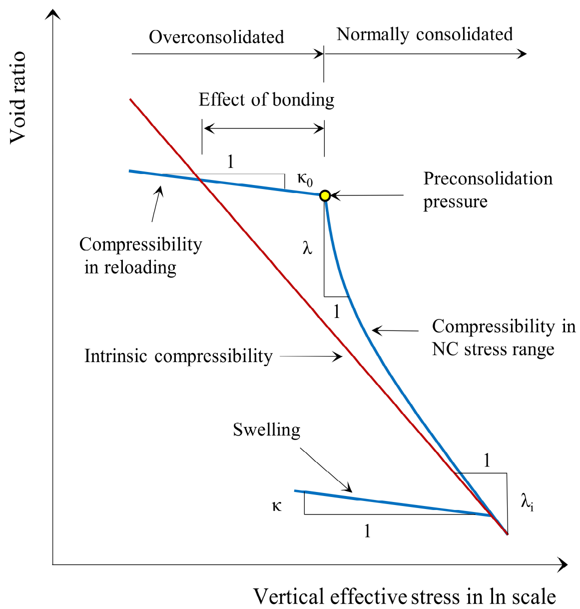


Figure 4. Parts of a semi-logarithmic plot of vertical effective stress and void ratio for a natural and reconstituted clay sample under one-dimensional compression. (Mataić, 2016).

2.6 Coefficient of consolidation

The consolidation velocity of soils is determined by the estimation of the coefficient of consolidation (C_v). Time-settlement calculations require this parameter, which can be defined from oedometer test using methods such as Taylor's square root method or Casagrande's log-time method. These methods are applied to time-settlement curves.

In Taylor's square root method, C_v is obtained for a degree of consolidation U=90% using Equation 10. The parameters in this equation are obtained from consolidation curves that are plotted with dial readings from the oedometer test, in millimeters, against the square root of time, in minutes (time-settlement curves). The procedure is found in Taylor's 1948 publication (Taylor, 1948).

$$C_v = \frac{0.848xd^2{}_{90}}{t_{90}} \tag{10}$$

Where d_{90} is the dial reading for a degree of consolidation U=90%
t_{90} is the time in minutes for the same degree of consolidation.

The C_v obtained by Casagrande's method corresponds to a degree of consolidation equal to 50%. Equation 11 is used in this method to obtain a C_v estimate by using a plot representing the deformation of the sample during the oedometer test as a function of the time logarithm. The description of the method and the parameters of Equation 11 can be found in Casagrande's 1936 publication (Casagrande, 1936). For each step load applied, there is a consolidation curve. This allows obtaining different C_v for different stress levels using either Taylor's square root method or Casagrande's method.

$$C_v = \frac{0.196xd^2{}_{50}}{t_{50}} \tag{11}$$

2.7 End of the primary consolidation EOP

The duration of primary consolidation is a function of permeability, compressibility properties, and maximum drainage distance (Feng, 2010). The end of primary consolidation is reached with the total dissipation of the excess pore water pressure. In an incremental loading oedometer test (ILOT), each loading step is maintained for 24 hours before the next one is applied. This means that the load is applied long enough for the soil to reach the end of primary consolidation EOP, which is assessed based on the measurements of excess pore water pressure or by the procedures proposed by Casagrande or Taylor (Casagrande, 1936; Taylor, 1948). The procedures by Casagrande and Taylor are applied to the deformation-time curve of each load step, which allows for determination of the EOP void ratio when the dissipation of excess pore is complete at a time t_p (Figure 5). In Casagrande's procedure, time in the deformation-time plot is represented in a semi-logarithmic scale, whereas the procedure proposed by Taylor requires plotting the strain versus the square root of time.

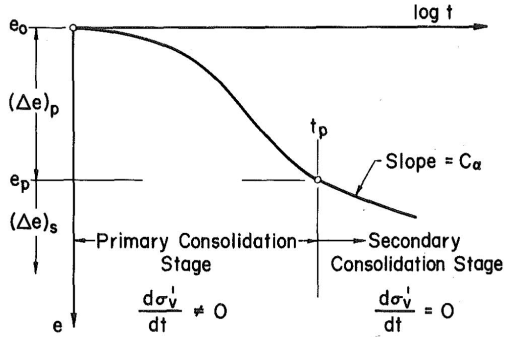


Figure 5. Definition of primary and secondary consolidation stages in a settlement curve. (Mesri & Choi, 1985).

In figure 5 the time t_p marks an inflection point after which, secondary compression takes place. The slope of the portion of the curve after this point is the coefficient of secondary compression (C_α) used to evaluate the secondary settlement of soil (Equation 12). There is an interrelationship between C_α and C_c, being C_c/C_α ratio constant for any soft clays (Terzaghi et al., 1996). However, many researchers have observed that C_c/C_α ratio is not constant for some soft sensitive clays (Graham et al., 1983; Leroueil et al., 1985; Yin et al., 2011; Mataić, 2016). The IL oedometer tests generally include data of both primary consolidation and secondary compression (Feng, 2010). In any case, when the values of C_α are required, secondary compression at different consolidation pressures can be allowed. It is possible to determine different C_α from different EOP e vs log σ'_v relationships considering that different primary consolidation times are obtained from different load steps. The expression for estimation of C_α is as follows (Mesri & Choi, 1985):

$$C_\alpha = \frac{\Delta e}{\Delta \log t} \tag{12}$$

Alternatively, C_α can be defined with respect to vertical strain instead of void ratio. Secondary settlement can be an important component of the total settlement in cases where t_p are small due for instance, to the presence of vertical drains. In normal field conditions, $t_p$is generally large with respect to the design life of the structures and the secondary consolidation is not an important factor. On the other hand, in cases where final consolidation pressure σ'_{vf} is just above σ'_p, t_p is small and C_α is large and increasing with time, and therefore, secondary settlement can be large and an important part of the total settlement (Mesri & Choi, 1985).

2.8 Methods for settlement calculations

The deformation behavior of clay under step loads of an oedometer test can be modeled by using models such as Janbu´s model (Janbu, 1970) and the compression index model. As it was mentioned earlier, soft Finnish clays do not behave in such a way that the compression range can be approximated to a straight line in semi-logarithmic scale. Therefore, the determination of a single coefficient of C_c and C_s is not always possible. Thus, for highly nonlinear compression behavior of soils, Janbu´s model is a more suitable model for settlement calculations. However, as the compression index method is suitable for Finnish silty clays and since it is widely used internationally, this method is also applied in the present study.

2.8.1 Compression index method

Settlement properties are described only as a single parameter in this model (C_c), which is determined by a straight line in the compression model. This means that this model only describes the settlement in the normally consolidated phase. The general form for this deformation model is described in Equation (13) (Terzaghi et al., 1996):

$$\varepsilon = \frac{C_c}{1 + e_o} \log\left[\frac{\sigma'}{\sigma'_{v0}}\right], \tag{13}$$

where $\sigma' = \sigma'_0 + \Delta\sigma_z'$, with σ'_0 being the in-situ effective vertical stress
$\Delta\sigma_z'$ is the increment in effective vertical stress

Similarly, the swelling-recompression part of the e vs log σ'_v can be approximated to a straight line whose slope is the swelling index *Cs*. Thus, Equation (13) can be written as (Terzaghi et al., 1996, p.108):

$$\varepsilon = \frac{C_s}{1+e_o}\log\left[\frac{\sigma_p'}{\sigma'_{v0}}\right] + \frac{C_c}{1+e_o}\log\left[\frac{\sigma'_{vf}}{\sigma_p'}\right] \tag{13a}$$

2.8.2 Tangent modulus method

Tangent modulus method uses the Ohde-Janbu deformation model to define the stress-strain relationship of very young and soft clays of post-glacial origin, which is defined in Equation (14) (Janbu 1970, p. 175). The calculation of deformations according to the tangent modulus method is divided into the normal consolidated and the overconsolidated part. For the normal consolidated part, the equations have the form of Equations (15) and (16) (Rantamäki et al., 1992, p. 211):

$$M = \frac{d\sigma'}{d\varepsilon} = m\sigma_v\left(\frac{\sigma'}{\sigma_v}\right)^{1-\beta} \tag{14}$$

$$\varepsilon = \frac{1}{m_1\beta_1}\left[\left(\frac{\sigma'}{\sigma_v}\right)^{\beta_1} - \left(\frac{\sigma'_{v0}}{\sigma_v}\right)^{\beta_1}\right] \quad for\ \beta_1 \neq 0 \tag{15}$$

$$\varepsilon = \frac{1}{m_1} ln\frac{\sigma'}{\sigma'_{v0}} \qquad for\ \beta_1 = 0 \tag{16}$$

Where $\sigma' = \sigma'_{v0} + \Delta\sigma_z'$, with σ'_{v0} being the in-situ effective vertical stress
$\Delta\sigma_z'$ is the increment in effective vertical stress
σ_v is reference stress of 100 kPa
m_1 is the modulus number of the normal consolidated part
β_1 is the stress exponent of the normal consolidated part

m_1 and β_1 are settlement parameters from oedometer tests where modulus number m_1 represents the compressibility of the soil and β_1 the form of the stress-strain curve of the normal consolidated part.

When $\beta_1 < 0$ the settling curve is concave on a semi-logarithmic scale, which is the case for normally consolidated clays. However, negative values of β_1 have been also observed for overconsolidated clays. For $\beta_1 > 0$, the form in convex, such is the case of settling curves of sand and silts.

The equation for the tangent modulus method and the parameters have to be selected according to the consolidation state of the soil either in the laboratory or in the field. For the overconsolidated part of the stress-strain curve, the settlement parameters are defined as m_2 and β_2. Thus, the values of m_1, β_1, m_2, β_2, have to be obtained in the following way:

β_1 and m_1 are parameters for normally consolidated part of the curve ($\sigma' > \sigma'_p$)
β_2 and m_2 parameters for overconsolidated part of curve ($\sigma' < \sigma'_p$)

With a curve fitting method, the parameters m_1, β_1, m_2, and β_2 can be estimated.

Equation 16 of the deformation model corresponds to the linear approximation of the relationship between the logarithmic effective vertical stress and the void ratio, which is the compression index model. This model is usually suitable for silty clays and lean clays. The relationship between C_c and the modulus number m_1 can be written as follows (Ravaska & Vepsäläinen, 2001):

$$m_1 = \frac{1 + e_0}{C_c} ln10 = \frac{2.303(1 + e_0)}{C_c} \tag{17}$$

Calculation of stress distribution under loading

The increment in effective vertical stress $\Delta\sigma_z'$ due to a load applied in both the compression index and tangent modulus method is calculated by the Boussinesq theory (Boussinesq, 1885). The stress distribution calculations are made based on the theory of elasticity by which, the soil mass is assumed to be semi-infinite, homogeneous, and isotropic. In settlement calculations, the procedure involves the calculation of the distribution of the additional stresses caused by the applied load. An embankment is an example of an applied load producing additional vertical stresses in the soil across the embankment.

3 Serviceability limit state verification according to Eurocode 7 (EC7)

3.1 Serviceability limit state (SLS)

Geotechnical structures such as embankments and footings must be designed to meet specific requirements in the form of physical conditions according to their planned use. For instance, excessive deformations in the supporting soil underneath a foundation can lead to a loss of serviceability even if structural failure is not involved. Thus, the serviceability limit state is a design approach that defines these conditions beyond which, the performance of structures is not acceptable with respect to their purpose of use. The verification of SLS involves checking that the limit established for those conditions is not exceeded during the service life of the structure. To carry out such verification in the case of ground-supported structures, the design values of the condition being checked has to be compared to pre-established allowable values by using the inequality in Equation (18) that is found on clause 2.4.8(1)P of EN 1997-1 (EC7) (CEN, 2004):

$$E_d \leq C_d \tag{18}$$

Where E_d is the design values of the effect of loads (e.g., settlements)
C_d is the limit value for serviceability

Verification that limit states are within allowable values is performed by one of the following methods: (i) by the partial factor method (ii) by using prescriptive methods (iii) by testing (iv) by observational method, and (v) by reliability-based methods (Estaire et al., 2019). The application of the partial factor method is described in EC7 (CEN, 2004). The objective of applying partial factors is to attain a target level of reliability in the design of geotechnical structures (Prästings et al., 2019). According to clause 2.4.8(1)P, the partial factor for SLS is equal to unity. This partial factor of 1.0 is applied to the characteristic values of the soil properties, service loads, or the effects of services loads, thus obtaining the design values to be used in the SLS verification. There is not a single way of combining factors between actions, soil properties, and resistances. EC7 (CEN, 2004) allows the selection of three different design approaches (DA). However, the selection of DA has only significance in ultimate limit states verification, since all partial factors in SLS are 1.0.

The limiting value for the service condition being considered during SLS verification should be specified during the design of the supported structure according to clause 2.4.8(5)P. In the absence of specific limiting values of structural deformations, Annex H of EC7 provides values that can be used as guidelines. It is worth mentioning that Annex H is purely informative.

3.2 SLS estimations in embankments design

In Section 12 *Embankments,* of EC7 (CEN, 2004), a list of the limit states to be checked during the design of an embankment is provided. Among the limit states related to the serviceability of embankments are settlements, cracking, creep displacements, and climate-related effects such as freezing, thawing, and extreme drying. In the case of embankments founded on soft clays, which are soils with high compressibility, the SLS verification should ensure that excessive settlements do not occur during construction (initial settlements). Time-dependent settlements due to primary consolidation and creep have to be taken into account, as well as deformation caused by changes in ground-water conditions. Clause 6.6.1 and 6.6.2 provide the principles for the calculation of settlements under an embankment founded on compressible soil. These include the three components of settlement to be considered and that have previously discussed in chapter 2 (initial, primary and secondary consolidation), and some considerations to evaluate the depth of the compressible soil layer under the embankment.

SLS estimations for the design of embankments involve the computation of initial settlements and long-term deformations (i.e., time-dependent deformations). Being initial settlements assumed as elastic deformations, their estimation is based on classical elastic theory. In the case of long-term deformations, estimation is usually carried out by applying Terzaghi's consolidation theory using the characteristic values of compressibility soil parameters (Meyerhof, 1995). A partial factor equal to the unity is applied to these characteristic values or on the characteristic values of deformations according to clause 2.4.8(1) (CEN, 2004). The estimates are then compared to allowable values of settlements according to the serviceability limits established for embankments.

In Finland, the Finnish Transport Infrastructure Agency (in Finnish: Väylä) establishes the allowable values of settlements occurring during the operation of roads for SLS verification. These values are functions of the road category. According to Väylä, verification of settlements aims to design and build structures whose settlements occurring during the operation of the road do not cause unplanned repair or maintenance measures. The values of vertical and horizontal settlements of the road, as well as longitudinal or cross-sectional differential settlements, must meet the limiting values set by the road category. The settlements for SLS verification are calculated for the operational period of the road (50 years usually). However, settlements occurring during the first 10 years of operation should be evaluated to verify that they are less than 40% of the total settlement attained over a period of 50 years. The recommended limit values for deflections are given in Table 4 of Chapter 6.3.2 of the guide "*Tien geotekninen suunnittelu*" (Liikennevirasto 2012).

The following chapter describes the definition of the characteristic values of ground properties that are currently used in the design and limit states verification of embankments following the corresponding clauses of EC7 (CEN, 2004). Following those definitions, characteristic values of compressibility parameters and deformations were obtained for the study cases of the present study. Likewise, following section 3.3, the definition of characteristic -and representative- values of soil properties according to the revised version of Eurocode 7, which CEN plans to publish in 2020, is also presented. This new version of EC7 is referred to as the October 2019 draft of EN 1997-1:2004 (EC7), or simply October 2019 draft.

3.3 Characteristic value according to EC7, 2004 version

EC7 part I in its clause 2.4.5.2(2) (CEN, 2004), defines the characteristic value of a ground property as *"a cautious estimate of the value affecting the occurrence of the limit* *state"*. The values of a parameter obtained through ground investigations and laboratory testing might differ from the true value of a ground property influencing the occurrence of the limit state, due to a series of factors. These factors include the extent and quality of the investigations, the number of samples and tests, natural variability of the ground, and the scatter of the test results (Frank et al., 2004). Accounting for these factors, a certain level of conservatism must be involved in the selection of the relevant parameters for design and the consequent limit states verification. For this reason, the characteristic value is addressed as a cautious estimate. Clause 2.4.5.2(4) enlists these factors as the main considerations in the selection of the characteristic value. Other factors enlisted are the existing knowledge of similar soil conditions, background information on the project, the soil volume involved in the limit state, and the ability of the structure founded in the soil to distribute the stresses throughout weak and strong zones (CEN, 2004). The amount of information collected in site investigations, as well as the background data and the scatter of the derived values, influence the amount of degree of confidence in the selection of the characteristic value. These factors are highly important as they influence the size of the margin between the true mean and the mean value of the test results if a mean value is governing the limit state. For instance, a considerable scattering of the results will mean greater uncertainty about the value governing the behavior (Frank et al., 2004).

Characteristic values selected as the value governing a limit state might be a cautious estimate of an upper or lower value of the soil parameter measured. It can be also a cautious estimate of the mean value around which, the test results fluctuate and that constitutes the most probable value (clause 2.4.5.2(5)). The selection of a value very close to the mean value or either the lowest or highest value of the measured soil parameter depends on the volume of the soil involved in a limit state and the associated design situation. Usually, the volume of soil that controls the occurrence of the limit state is greater than the volume that is tested in ground investigations and laboratory tests, which is only a small portion of the total volume (Orr, 2017). Unless a special case of local failure or weak points governing the limit state is being considered or detected, both locally weak and strong soil will be involved in failure. In this case, failure will take place on an extended surface, and therefore, the test results can be averaged, as there is compensation between weak and strong soil zones (Prästings et al., 2019). The value governing the limit state, in this case, will be the mean value of the soil parameter and the characteristic value should be a cautious estimate of it (CEN, 2004).

Besides providing some guidelines in the selection of this cautions value, EC7 also addresses the statistical estimation of the characteristic value. Statistically, the characteristic value of a soil parameter corresponds to a specific fractile of the density distribution function (Orr, 2017). A fractile is a cut-off point defining a certain probability in the distribution function. Thus, the characteristic value reduces to a certain percentage, the probability of occurrence of a less favorable value of the soil parameter during the service time of the geotechnical structure. Selection of the fractile as the characteristic value of a ground property must satisfy the clause 2.4.5.2(11): *"If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%"* (CEN, 2004). According to further clarification under this clause, two values satisfy this statement. On the one hand, the 5% lower fractile of the probability density function (PDF), which reduces to 5% the probability of having a lower unfavorable value governing the limit state than the characteristic selected. This low value applies in the case of local failure, where a small soil volume is involved in the limit state. An example of a design situation where local failure is considered is seepage (Orr, 2017). On the other hand, the characteristic value in soil profiles with a continuous spatial variability and no local weak zones is an estimation of the mean value of the parameter governing the limit state with a confidence level of 95%. This estimation will result in a value that is more favorable than the mean value of the soil parameter with 95% reliability. Thus, the cautious estimate of the mean value in non-local failure situation also satisfies the statement of clause 2.4.5.2(11). Figure 6 shows the PDF of the derived values of a soil property and the PDF of the mean value ($\bar{X}$) of derived values with the fractiles corresponding to the characteristic values contemplated in EC7.

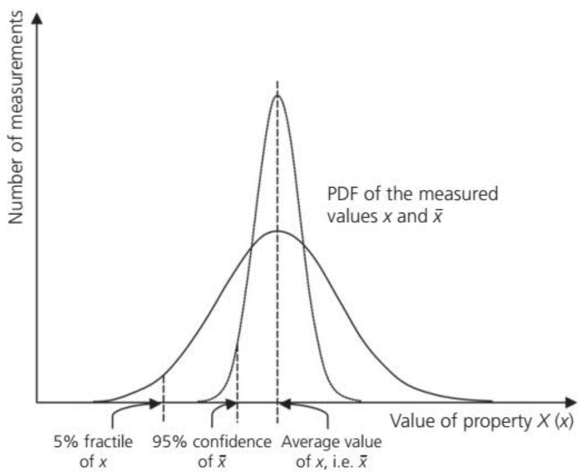


Figure 6. Characteristic values calculated as fractiles of the PDF. (Prästings et al. 2019).

3.4 Characteristic, representative and nominal values definition according to October 2019 draft

3.4.1 Characteristic value definitions

In section 4.3.2 *material and product properties* and the informative Annex B *characteristic value determination procedure*, the October draft gives an insight into the definition and selection procedure of the representative, nominal and characteristic values of ground properties to be used in SLS verification. The terms representative and nominal values applied specifically to ground properties are new proposed additions of the October draft to the Eurocode 7. Likewise, a redefinition of the characteristic value is given, along with a procedure for its estimation using a statistically based equation. According to the draft, the representative value of a ground property is the value affecting the limit state, a definition that is currently reserved for the characteristic value of a ground property in EC7 published in 2004 (CEN, 2004). This means that representative values of ground properties will be acknowledged as the values to be used in ultimate limit state (ULS) and SLS verifications, rather than the characteristic value as defined currently.

As it was the case for the characteristic value of ground properties defined in EC7 (CEN, 2004), representative values *“shall be based on the derived values determined in the ground investigation*” and can be estimated either as a cautious estimate based on experience or by statistical methods. However, the selection method will define whether the representative value is estimated from the nominal value or the characteristic value. Thus, if the value affecting the occurrence of the limit state is selected based on judgment and experience, the representative value will be estimated using the nominal value as defined in Equation (19). The selection of statistical methods, conversely, will define the representative value as a function of the characteristic value (Equation 20).

$$X_{rep} = \eta X_{nom} \tag{19}$$
$$X_{rep} = \eta X_k \tag{20}$$

Where η is a conversion factor
X_{nom} is the nominal value of the ground property, selected as a cautious estimate of the value affecting the occurrence of the limit state.
X_k is the characteristic value, determined by statistical methods.

The conversion factor η accounts for scale effects, effects of moisture and temperature, effects of aging of materials, and other parameters. In geotechnical design, the value of the conversion factor is equal to the unity, unless that EC7 (CEN 2004) or the National Annexes indicate a different value (Estaire 2019).

EC7 provides a list of the factors that need to be taken into account when selecting the characteristic value. Similarly, a list of factors to consider is presented in the October 2019 draft for the selection of the representative values, but factors such as the spatial and statistical variability are addressed as sources of uncertainty, a term that was not used in EC7 (CEN, 2004) in the definition of the characteristic value. In fact, the draft in the Annex B *characteristic value determination procedure* provides more information about the uncertainties involved in the selection of characteristic geotechnical parameters. Factors to be taken into account in the October 2019 draft are as follows:

- *pre-existing knowledge including geological information and data from previous projects.*
- *uncertainty due to the quantity and quality of site-specific data.*
- *uncertainty due to the spatial variability of the measured property; and*
- *the zone of influence of the structure at the limit state being considered*

In order to establish the value affecting the occurrence of the limit state, EC7 states the need to determine the total volume of soil within which, failure is developed. As it was explained before, this is related to the concepts of local and global failure. Local failure situation, involving a small portion of weak soil, requires the cautious estimate of the low value of the property whereas, in the case of global failure concerning a large portion of the soil, the averaging of the soil property is assumed (Orr, 2017, Prästings et al., 2019). However, October 2019 draft uses the concept of spatial variability to distinguish between the scenario in which, the mean value can be selected as the parameter governing the limit state and when the designer should select a cautious estimate of inferior or superior value as the characteristic value. According to the clause, the selection is done depending on the sensitivity of the verification of the limit state to the spatial variability of the property. If the verification is sensitive to spatial variability, the characteristic value must be selected as an inferior or superior value of the ground property.

Although it is not explicit in the draft, a verification of the limit state being sensitive to spatial variability means that high local variation of a soil property is causing the presence of a weak zone where a low or upper value can result critical. In this case, the scale of fluctuation of the soil property is high with respect to the extent of the zone under a governing failure. Therefore, no reduction of the contribution of the spatial variability to the uncertainty in the property can be performed. This situation describes the case of local failure and therefore, a more conservative value of the ground property must be estimated as the characteristic value, either the lower or upper value. On the other hand, when failure takes place within a large volume of soil where averaging is possible, the contribution of spatial variability is not significant and its accountability for the uncertainty in the soil property can be reduced (Prästings et al., 2019), leading to a less conservative estimation of the characteristic value.

Figure 7 shows a comparison in the definition of characteristic value between the EC7 2004 version and the October 2019 draft. Likewise, the relationship between representative, nominal, and characteristic value for the assessment of ground properties is presented. According to Equation 20, the representative value can be essentially the same as the characteristic value but only when determined by statistical methods and taking into account the conversion factor. This represents then, a shift in the definition of the characteristic value and its use in SLS verification.

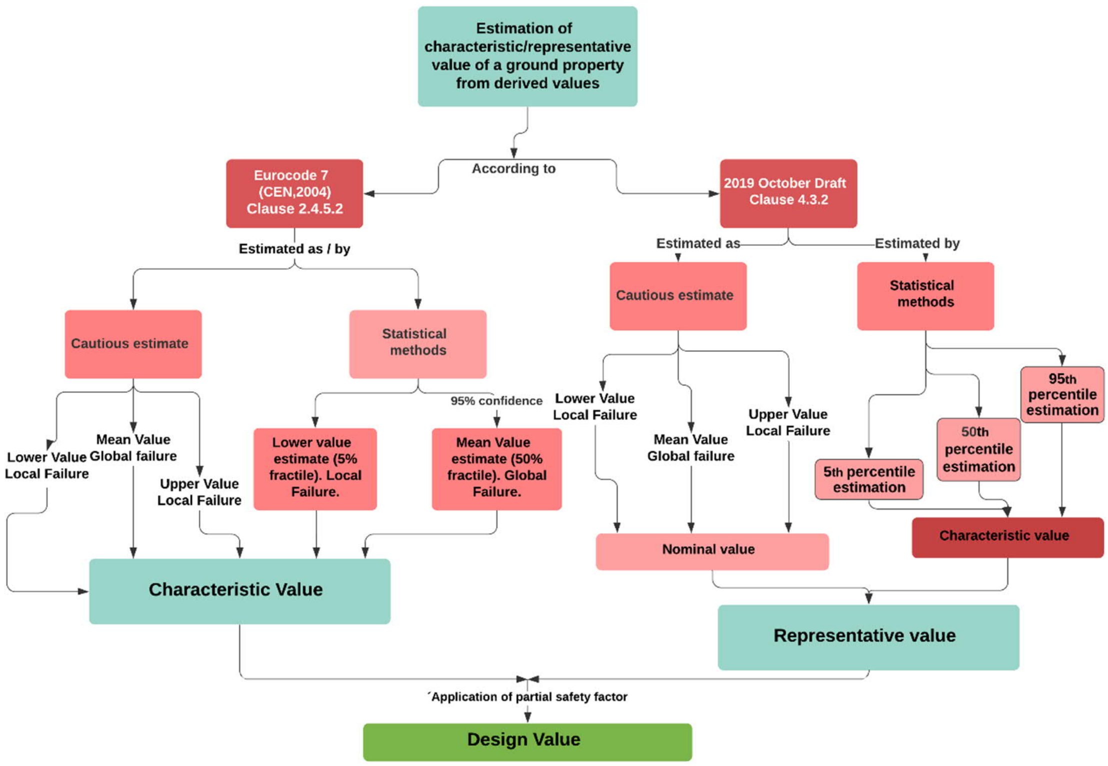


Figure 7. Comparison between the definition of characteristic, representative, and design value according to EC7 and October 2019 draft.

3.4.2 Characteristic value estimation according to October 2019 draft

In October 2019 draft, determination of the characteristic value necessarily involves the use of statistical methods, restricting the cautious estimate based on judgment and experience only to nominal values as explained above. This redefinition of the characteristic value might give a more consistent and objective selection of this value. However, the subjective judgment involved in the selection of ground property value will remain if a nominal value is used for design. According to clause 4.3.2, the characteristic value has to be determined statistically with a confidence level of 95%. According to the October 2019 draft, the characteristic value to be used in geotechnical design can be an estimate of one of the following three percentiles from the probability density function of the ground property, assuming a normal distribution of the data:

1. The mean value (X_{mean}), which is the 50% fractile (50^{th} percentile) of the ground property.
2. If an inferior value is used ($X_{k,inf}$), this must correspond to the 5^{th} percentile of the distribution of the ground property. The selection of this percentile will reduce to 5% the probability of having an unfavorable value lower than the characteristic value.
3. If a superior value is selected ($X_{k,sup}$), the characteristic value corresponds to an estimate of the 95^{th} percentile. This superior value has a 95% probability that the real value of the property is lower, and 5% of being exceeded.

Figure 8 shows the PDF of a ground property X from which, the percentiles that correspond to the characteristic values X_{mean}, $X_{k,inf}$ and $X_{k,sup}$ described above, are calculated. The superior value $X_{k,sup}$, was not explicitly regarded as the 95^{th} percentile in EC7 and it is excluded from clause 2.4.5.2(11) as a value whose probability of being violated by an unfavorable value is not higher than 5%. However, it is included in the October 2019 draft as there are situations where an upper value selected as characteristic value gives a more conservative result, such as the case of downdrag (Frank et al., 2004).

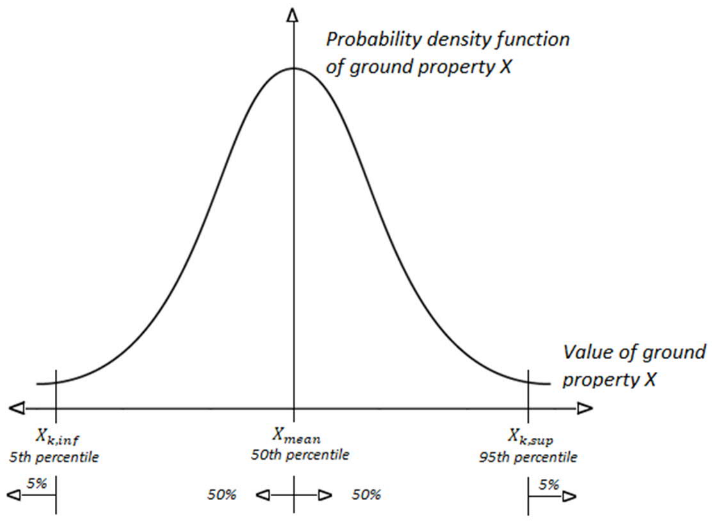


Figure 8. Percentiles corresponding to the characteristic values as defined by October 2019 draft.

For derived values of the ground property *X* following a normal distribution, section 4.3.2 of October draft provides a statistically based equation (Equation 21) for the assessment of the characteristic value of ground properties (X_k), that allows obtaining estimates of any of the percentiles described in the figure above, depending on the design situation. The equation satisfies then, the description that is given to the characteristic value in clause 2.4.5.2: "*If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%*".

$$X_k = X_{mean}[1 \pm k_n V_x] = X_{mean}\left[1 \pm \frac{k_n \sigma_x}{X_{mean}}\right] \tag{21}$$

Where V_x is the coefficient of variation of the ground property that is selected. according to three different cases that will be explained below.
X_{mean} is the mean value of the derived values measured in the laboratory and ground investigations.
k_n is a coefficient that depends on the number of sample derived values (n) used to calculate X_{mean}.
σ_x is the standard deviation

V_x also known as the sample coefficient of variation, is obtained by dividing the sample standard deviation by the mean of the derived values of the ground property. V_x quantifies the relative dispersion of the data around a central trend exhibited by the variation of the ground property (Uzielli et al., 2006). Thus, V_x accounts for the uncertainty related to the inherent variability of the soil property.

The October draft document provides a set of tables to select the value k_n as function of the number of tests that applies to the case under which, V_x is determined and based on the percentile selected as X_k (see: Appendix I). The sample mean value of the soil property determined from soil measurements differs from the true mean value of the soil property because of the limited number of soil samples (Schneider, 1997). This causes a sampling uncertainty, which k_n accounts for. With a higher number of soil measurements, the sampling uncertainty is reduced and k_n decreases, which leads to a less conservative estimation of the characteristic value. On the other hand, when soil data is limited, k_n is larger and so is the contribution of sampling uncertainty to the total uncertainty. Coefficient k_n together with V_x account for the total uncertainty of the soil property. Thus, the total uncertainty in Equation 21 can be divided into sampling uncertainty and inherent spatial variability uncertainty.

The symbol $\pm$ allows calculating either an estimate of the 5^{th} or 95^{th} percentile of the ground property, depending on what value of X_k (inferior or superior) yields more conservative results. Thus, (-) shall be used if a lower value of X_k is critical and (+) if an upper value is critical. The same principle applies when calculating the cautious estimate of the mean value as the characteristic value. When the estimate X_k below the true mean of the ground property is more conservative, (-) should be used, whereas (+) is applicable when the critical value is above the true mean.

The three different cases under which, V_x can be selected, are:

- Case 1: "V_x known". Only to be selected when the coefficient of variation of the soil parameter has been determined previously. For instance, when it is determined from site-specific data or when it is selected from a previous estimation made for a comparable situation.
- Case 2: "V_x assumed". This case is selected when the coefficient of variation to be used is chosen from tables of indicative values of V_x for different ground properties.
- Case 3: "V_x unknown" should be used when the coefficient of variation of the ground property being determined is unknown ab initio. In this case, a detailed procedure is also provided, although not recommended.

The values of k_n corresponding to each of the cases under which V_x is selected are presented in Annex B of the October 2019 draft. This annex is solely informative. Appendix I presents Tables B1, B4, B5, B6, and B7 from October 2019 draft. Table B1 contains the equations for the determination of k_n values for different cases and types of estimation that have been previously described in this chapter. Tables B4 to B7 contains the values of k_n as functions of the number of samples (n). The minimum value of n according to Tables B.4 to B.7 is 2.

4 Study cases

4.1 Haarajoki test embankment

4.1.1 Background

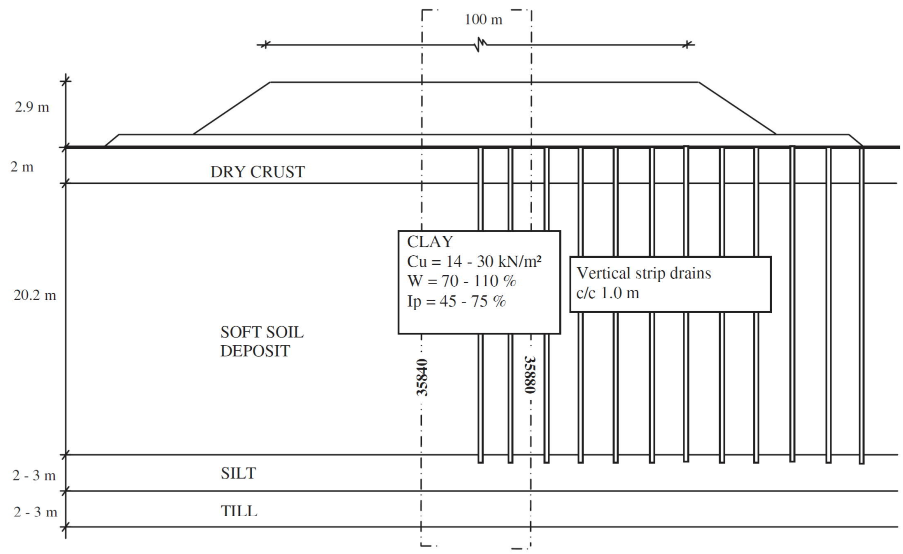
Haarajoki test embankment was built and instrumented in 1997 near Järvenpää, Finland, by the Finnish National Road Administration. The purpose of the test embankment was to evaluate the accuracy of settlement calculations by conventional and advanced calculation models (Kinani et al., 2001). The settlement predictions were part of an international competition where the participants had to predicted settlements, horizontal displacements, and changes in pore pressure (Lojander & Vepsäläinen, 2001). Extensive soil investigations and a large number of laboratory tests were performed to provide the participants with the required soil parameters to carry out the tasks. The instrumentation used for field measurements and observations included inclinometers, piezometers, pressure cells, and settlement plates (Yildiz et al., 2009). Lojander and Vepsäläinen (2001) published the results of the competition.

Haarajoki test embankment is founded on soft soil deposits, which exhibit a high degree of anisotropy and natural interparticle bonding influencing the stress-strain behavior, the permeability, and the shear strength of the deposits (Yildiz et al., 2009). Half of the soil deposit upon which the embankment was built, contained vertical drains to speed up the consolidation process. Virgin soil layers compose the other half of the deposits under the embankment with no soil improvement (Lojander & Vespsäläinen, 2001). Field observations showed that primary consolidation of the half with vertical drains was still going on after three years of the embankment construction (Vepsäläinen et al., 1997).

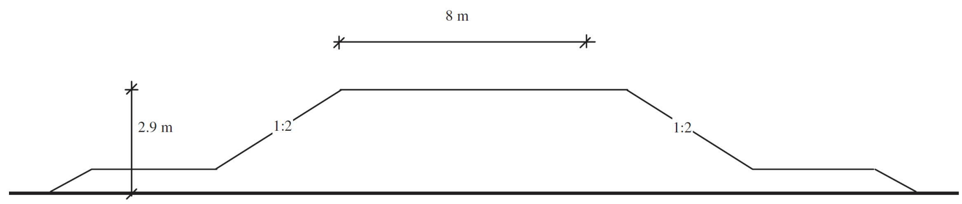
4.1.2 Embankment geometry and soil profile

Haarajoki test embankment is 2.9 m high and 100 m long, with a crest of 8 m wide and slopes of gradient 1:2 (Yildiz et al., 2009). Figure 9 shows the longitudinal profile and cross-section of the Haarajoki test embankment. The groundwater level is at the ground surface. The embankment material has a density of 21 kN/m^3 and is composed of sandy gravel and gravel. Under the embankment, there is a 2 m thick dry crust layer, which is highly overconsolidated. The dry crust layer is followed by a soft soil layer of 20.2 m thick, which is slightly overconsolidated. Beneath the soft soil deposit, 2- 3 m thick silt, and till layers are found.

The water content of the dry crust layer varies between 35 - 55%. The low water content led to desiccation, which might cause swelling of the soil. In turn, there is a reduction in void ratio and consequently, an increase in unit weight. Due to these characteristics, a low compressibility of the dry crust is expected. On the other hand, the water content of the soft clay deposit is significantly higher with values within a range of 70-120%. The unit weight of these deposits varies between 13.72 - 16.21 kN/m^3 whereas the values for the dry crust fall in the range of 16.86 - 17.68 kN/m^3. Some of the soil properties of Haarajoki embankment from field and laboratory results are summarized in Figure 10.

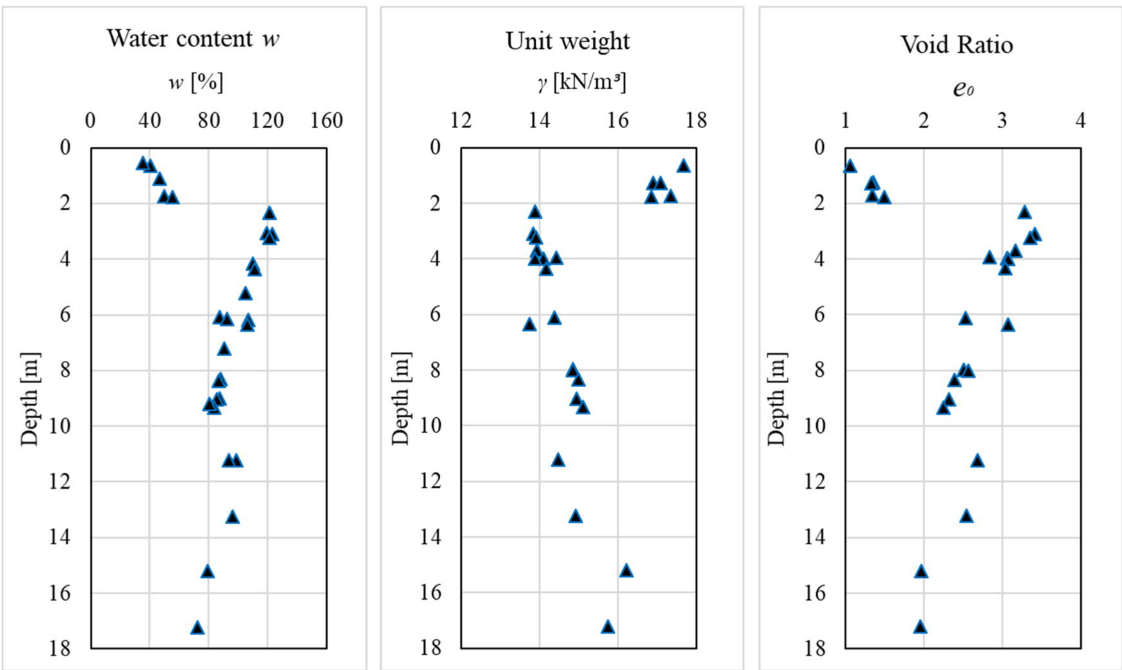


(a)



(b)

Figure 9. Haarajoki test embankment: (a) longitudinal section; (b) cross section. (Yildiz et al. 2009).



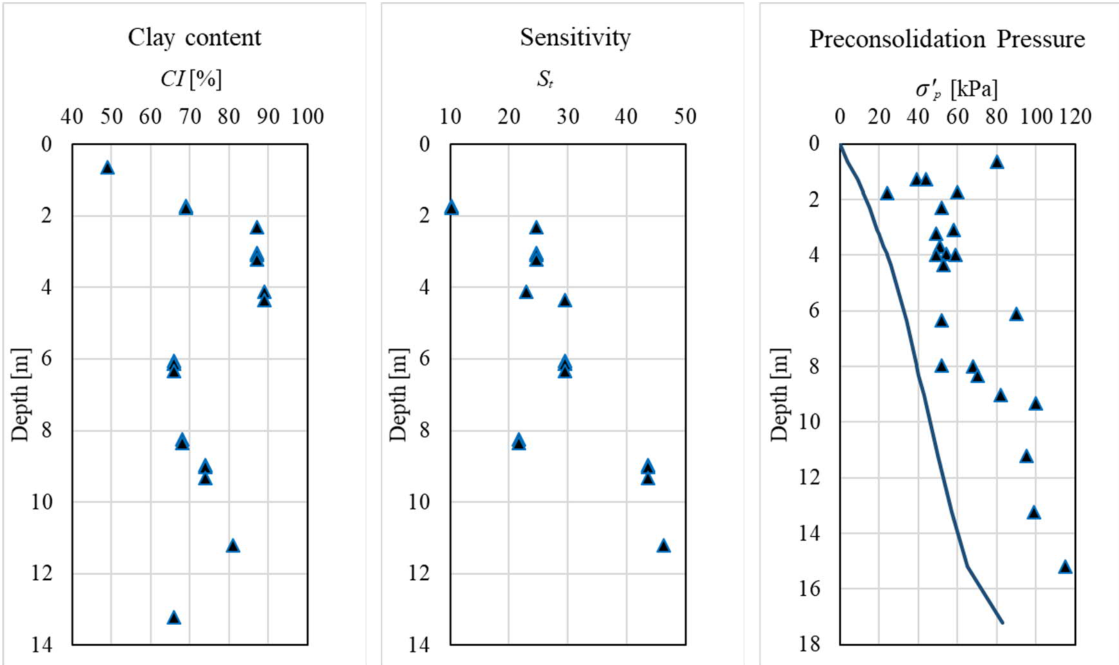


Figure 10. Typical soil properties in the Haarajoki test embankment case.

4.1.3 Haarajoki test embankment monitoring

Based on data from field observations, the most compressible region of the soil deposits under the Haarajoki embankment is located at a depth of 2 – 4 m. Likewise, the effect of the vertical stress increase due to the embankment load is not significant after 10 m depth (Amavasai, 2015, p.9). During the first two years after construction, the side of the embankment built on the virgin soil layers had a settlement of 372 mm whereas the side built on vertical drains had a settlement of 633 mm (Lojander & Vepsäläinen, 2001, p.13). Länsivaara (2001, p.40-41) reported additional observed values of settlement at 3.5 years, during which a settlement of approximately 400 mm was attained in the virgin soil layers while the side with vertical drains had a settlement of around 700 mm.

4.2 Kujala embankment

4.2.1 Background

Kujala embankments were built and instrumented in 2017 for the Kujala interchange of Highway 12 located in Southern Ring Road of Lahti, Finland. The high sensitivity of the silty layers composing the subsoil in Kujala area caused a high uncertainty in the derivation of soil parameters, which was reflected in the low quality of oedometer tests. In consequence, two 4-5.5 m high test embankments were constructed aiming to improve the settlement predictions by applying the observational method. The embankments were instrumented with settlement plates, inclinometers, pore pressure probes, and fluid hose settlement measurement devices. The embankments remained as part of the ramp structures of Kujala interchange. The collection of the monitoring data started six months before the construction of the interchange and continued for almost 1.5 years. During this time, settlements, horizontal displacements, and pore pressure were measured. (Löfman

& Korkiala-Tanttu, 2021). Comprehensive ground investigations were carried out in addition to monitoring.

4.2.2 Embankment geometry and soil profile

The Kujala test embankment selected for this study is on average 4 m high and has a crest width of 18 m. The test embankment is approximately 20 meters long. The slopes of the embankment were constructed at a slope of 1:1.5. The fill material of the embankment is crushed rock with a 0-90 mm size and has a density of 20 kN/m^3. The embankment is founded in approximately 12 meters of clay and silt layer, which is mostly overconsolidated. The fine-grained soil layers and the bedrock are separated by a thick layer of moraine. For the subsoil under the selected embankment, the depth of the groundwater level is around 2 m. The dry crust is stiff and has a thickness of 2 m. The cross-section of the embankment is shown in Figure 11.

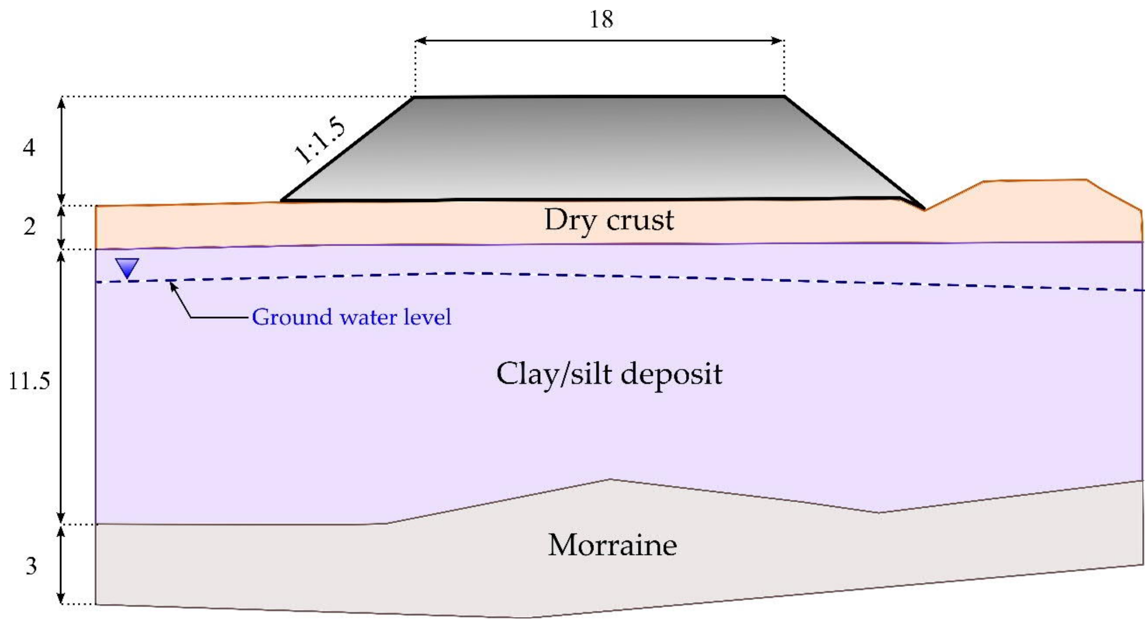


Figure 11. Kujala test embankment cross-section. Units: meters.

The water content of the clay and silt layers varies between 30 – 76%. The unit weight of these fine-grained layer fluctuates between 16 – 19 kN/m^3. The clay fraction ranges from 30 – 55 %. According to the laboratory tests, the OCR of the clay/silty layers is 1 – 2. Sensitivity is especially high for the silty layers. The dry-crust layer is highly overconsolidated with an OCR that ranges from 4 – 6. Some of the soil properties of the Kujala embankment from laboratory results are summarized in Figure 12. Preconsolidation pressure in Figure 12 was obtained from oedometer tests.

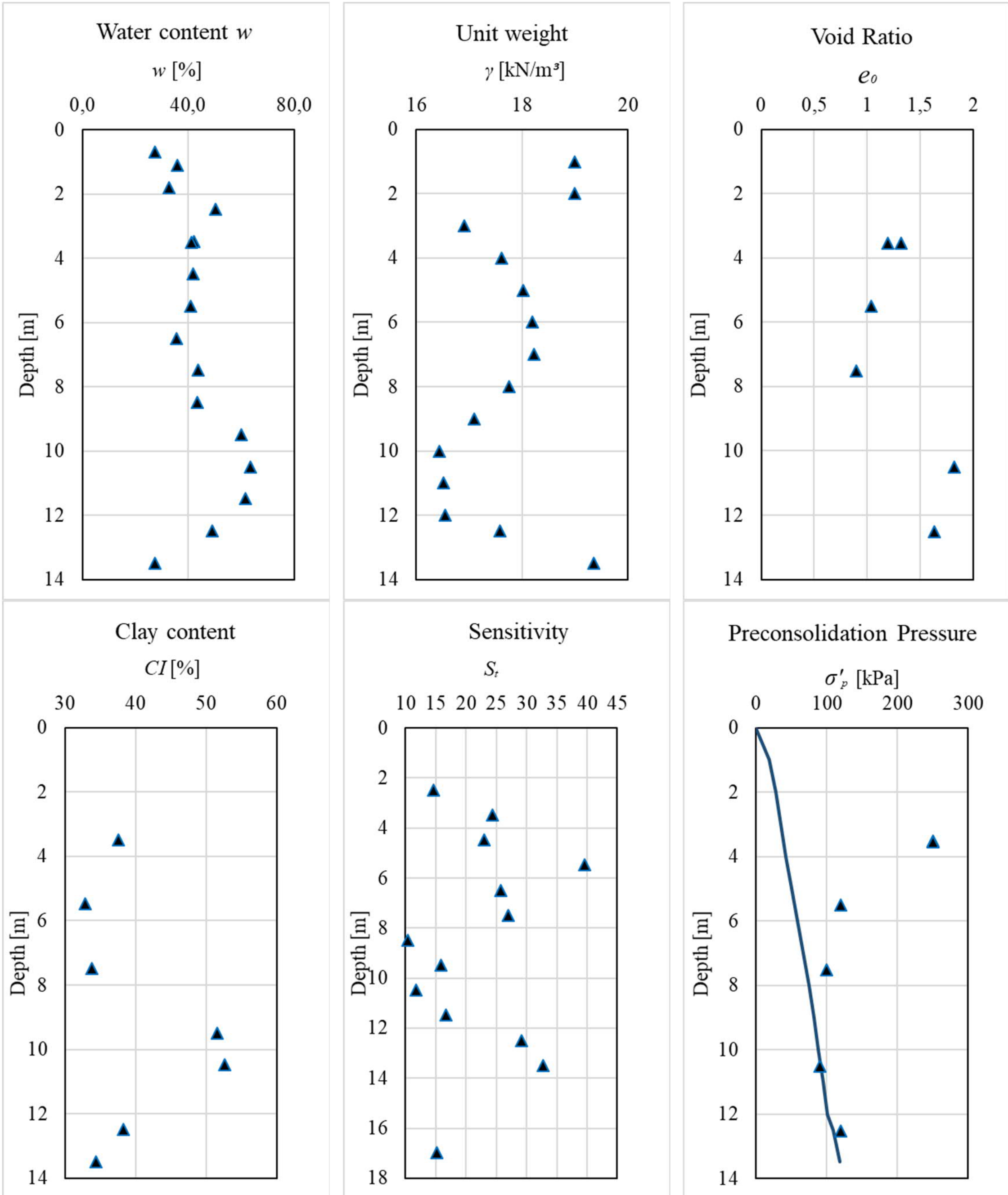


Figure 12. Typical soil properties in the Kujala interchange area.

4.2.3 Monitoring in Kujala area

During the 1.5 years of monitoring of the Kujala embankments, the observed settlements were 65-115 mm, with most of the primary consolidation attained over this period. Predicted values (100-1000 mm) were significantly above the observed values (Löfman & Korkiala-Tanttu, 2021). The low quality of the oedometer tests led to an underestimation of the preconsolidation pressure with a subsequent overestimation of the predicted settlements, which were adjusted based on the monitoring data and ground investigation results. Thus, the values of preconsolidation pressure shown in Figure 12 may not reflect the actual stress state of the soil. Soil parameters from the calibrated settlements in the Kujala case were used for settlement computation in this thesis. The values are shown in Chapter 5.2.

4.3 Murro test embankment

4.3.1 Background

Murro test embankment was built in 1993 near Seinäjoki, Finland by the Finnish Road Administration. The embankment construction aimed to model the behavior of soft soil for future improvement of the design of road embankments built on soft soil (Koskinen et al., 2002). It is founded on a natural soft clay deposit about 23 m deep. The deposits are characterized by a high degree of anisotropy and destructuration. The embankment construction was complete within two days and it was instrumented with settlement plates, inclinometers, extensometers, and pore pressure probes. Since its construction in 1993, the embankment was monitored until the year 2007, when the last measurement was taken (Karstunen & Yin, 2010). Extensive experimental data from oedometer tests are available for this case.

4.3.2 Embankment geometry and soil profile

An illustration of the Murro test embankment cross-section is shown in Figure 13. The test embankment is 2 m high and 30 m long with 1:2 slopes. The crest of the embankment is 10 m wide. The fill material of the embankment consists of crushed gravel with 0-65 mm grain size and a density of 20 kN/m^3. The groundwater level is estimated to be at 0.8 m below ground level.

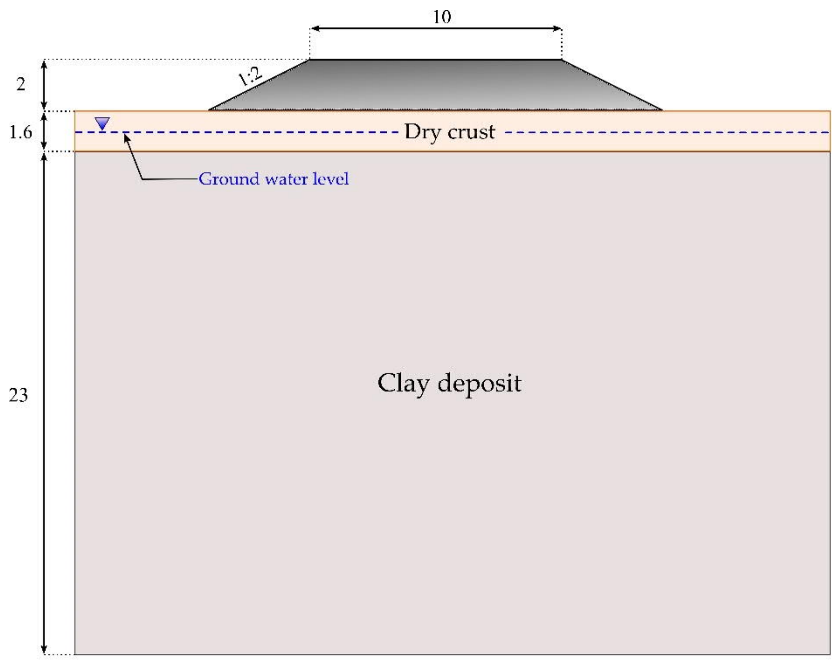


Figure 13. Murro test embankment cross-section. Units: meters.

Murro test embankment is underlain by a 1.6 m of heavily overconsolidated dry crust. This dry crust is followed by a 23 m thick clay deposit, which is normally consolidated. Based on classification tests, the deposit is classified as either silty clay or clayey silty, with clay content values below 30%. The water content in the first 10 meters beneath the embankment varies between 65 and 100% and remains almost constant for depths below the 10 m. The sensitivity is moderate with values between 2 and 14. Figure 14 shows some index properties of the subsoil in the Murro embankment site.

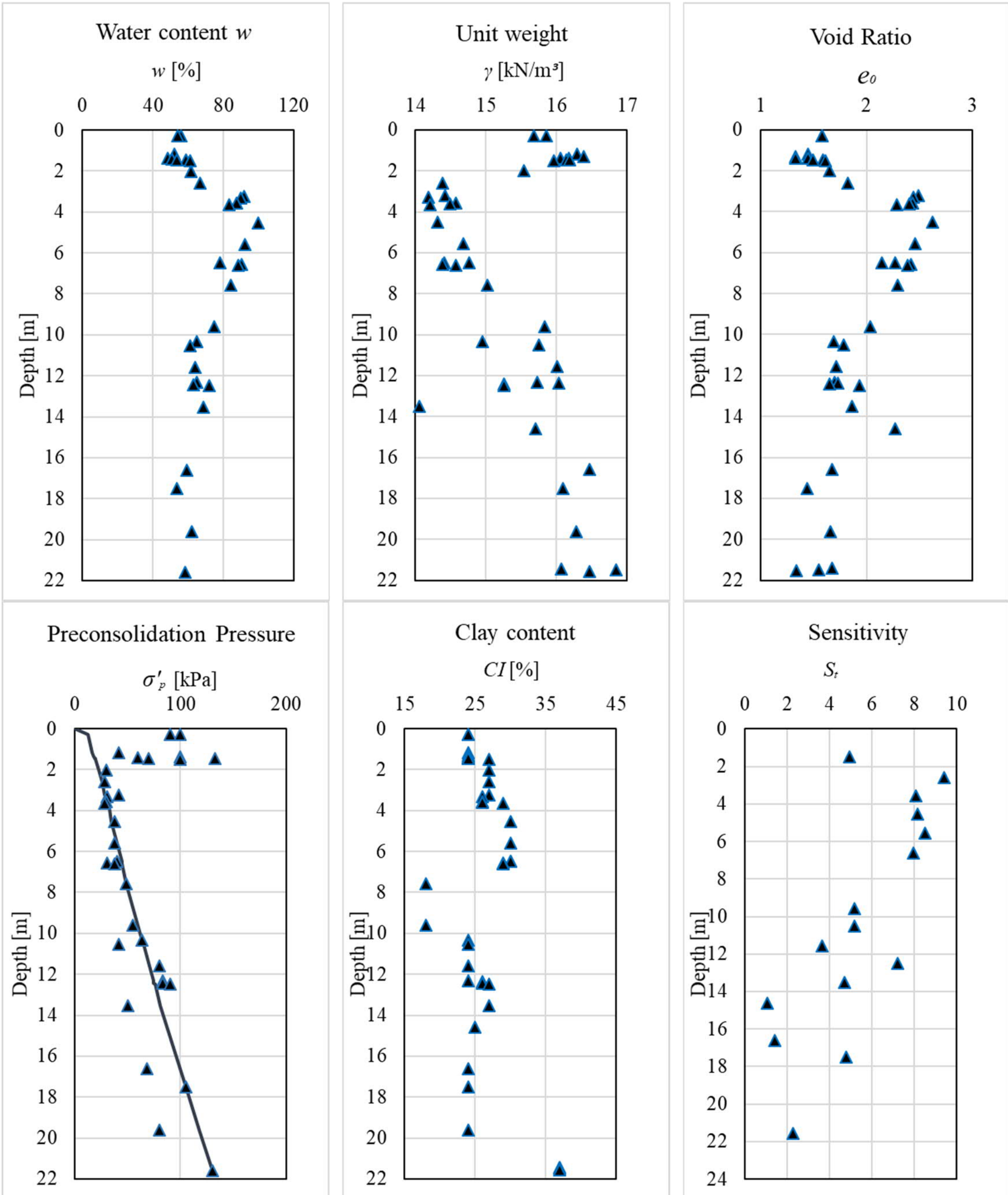


Figure 14. Typical soil properties in the Murro site.

4.3.3 Monitoring of Murro test embankment

Observations of settlements, horizontal displacements, and pore pressure are available for a period of 8 years after construction. The observed value of total settlement over this period is 798 mm, and the maximum horizontal displacement is 123 mm. The total settlement occurred at a depth between 0 – 8.5 m (Koskinen et al., 2002), and the most compressible region is at a depth between 1.6 – 6.7 m (Karstunen & Yin, 2010).

4.4 Östersundom test embankment

4.4.1 Background

Östersundom test embankment is located in Östersundom district, in Eastern Helsinki, Finland. The deposits in the Östersundom area are predominantly clays whose thickness varies between 3 – 16 m along with rocky areas. The area has been part of all the late glacial and post-glacial stages of development of the Baltic Sea, which has contributed to the formation of the region's soil. (Kosonen et al., 2015, p. 7). The test embankment was built by the geotechnical division of the city of Helsinki to evaluate settlements and the feasibility of construction projects in the area. The first part of the test embankment was built in March 2014 and the second one in December 2014. Monitoring and survey in the area of the embankment have been performed over the period 2012 – 2014. The instrumentation included settlement plates installed at different depths to measure the settlement in different layers.

4.4.2 Embankment geometry and soil profile

The embankment was built on top of a geosynthetic reinforcement installed on the ground, at a level of about +2.4 m above the sea level. The height of the lower part of the test embankment is about 0.4 m, and the height of the central part has been about 0.8 m. Thus, the total height of the embankment is 1.2 m and 21 m long. The top part of the embankment is 10 m wide. The groundwater level is estimated to be at a depth of 0.6 m above the ground. Figure 15 shows the cross-section of the embankment.

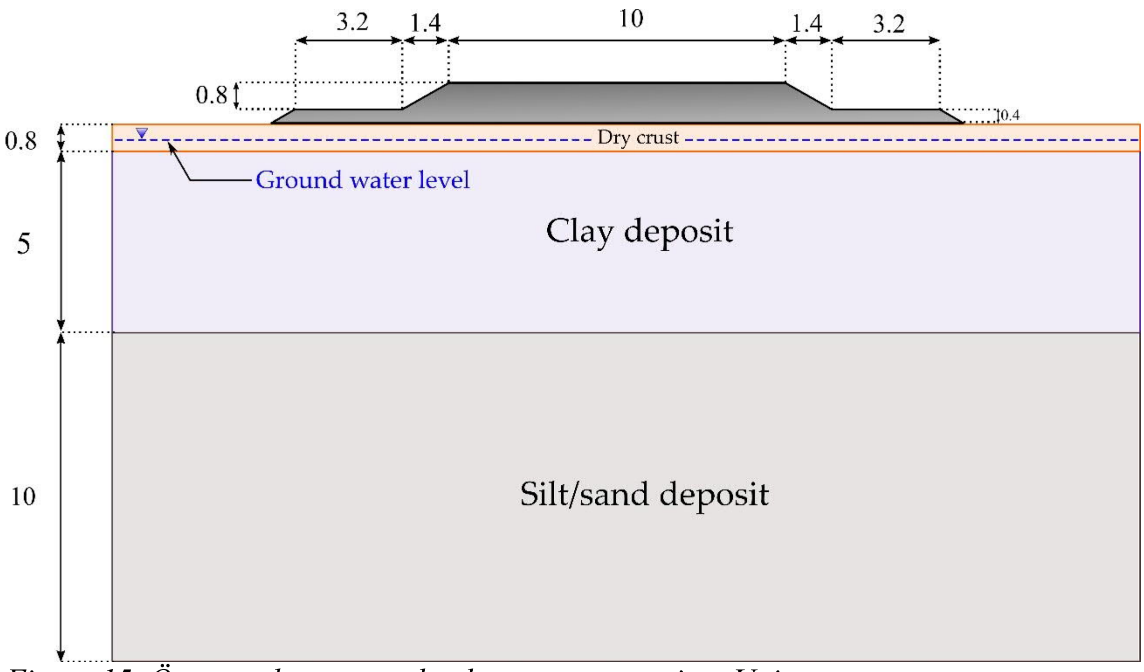


Figure 15. Östersundom test embankment cross section. Units: meters.

A heavily overconsolidated dry crust layer underlies the embankment. The dry crust layer is 0.8 m thick. Following the dry crust, there is about 5 m thick soft clay deposit, which is slightly overconsolidated. Under these clay deposits, there is a layer of silt and sand about 10 m thick. Several laboratory and field investigations are available for this case. Laboratory tests include classification tests, oedometer tests, and triaxial tests. The typical soil properties determined from the results of these tests are shown in Figure 16.

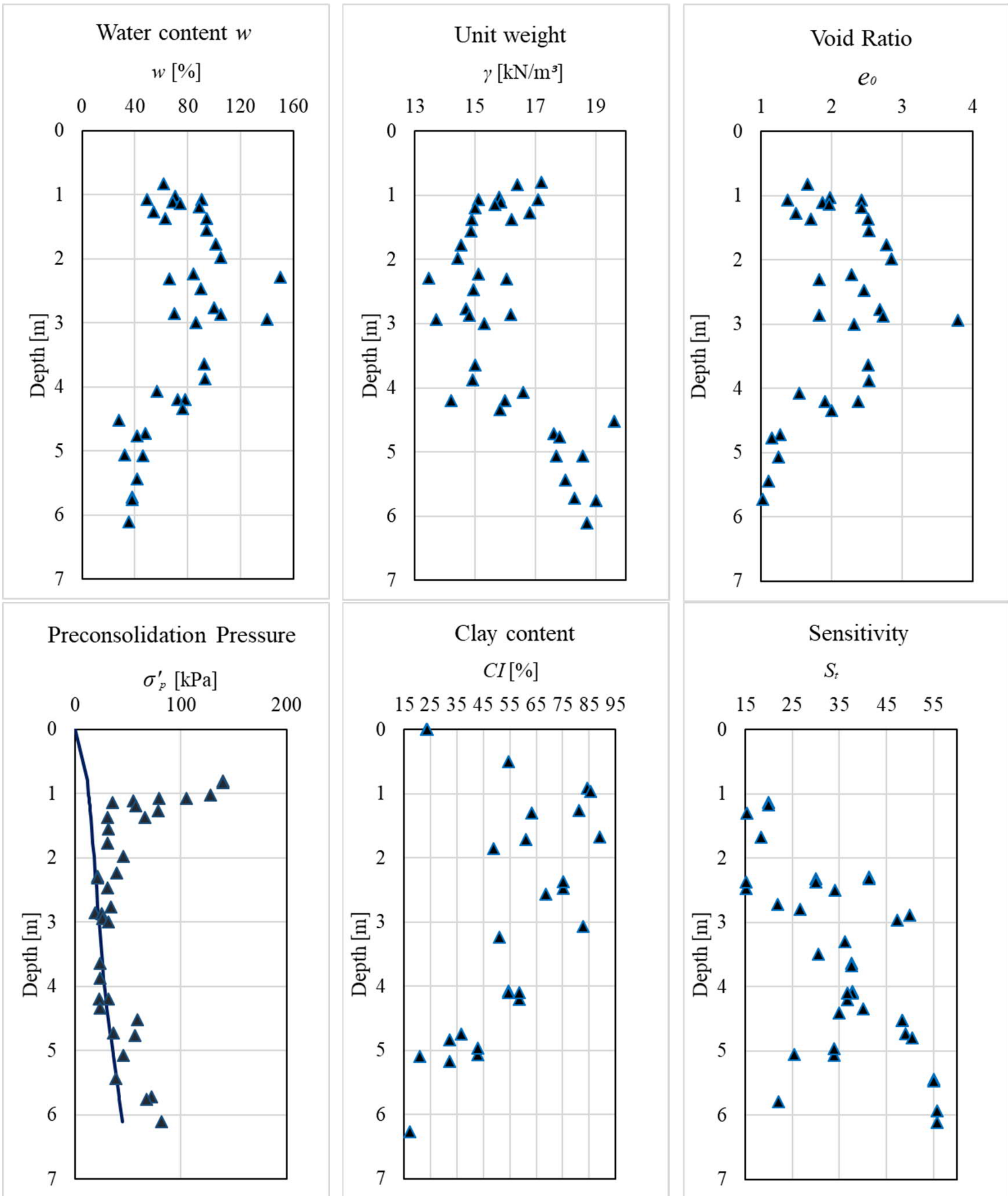


Figure 16. Typical soil properties in Östersundom site.

4.4.3 Monitoring of Östersundom test embankment

According to the values reported by Köylijärvi (2015), observed values of settlement for Östersundom test embankment over 1.5 years is about 125 – 300 mm. The end of primary consolidation was not yet attained at the time of these observations.

5 Methodology

This chapter presents the methods used in this thesis to carry out the computation of primary consolidation settlements for each of the cases described in Chapter 4. Section 5.1 presents the procedure for the selection of soil parameters and the statistically-based equations used to estimate the representative/characteristic values. Section 5.2 discusses the settlement calculation methods used in each case to compute total settlements and Section 5.3 explains the procedure to carry out a time-settlement analysis for one of the selected cases where the use of characteristic values is also assessed for the coefficient of consolidation of each clay layer.

5.1 Determination of best estimate and characteristic values of soil properties

Computation of total settlements for each of the study cases described in Chapter 4 was divided in two main analysis: (1) total settlement from unmodified parameters (best estimate of the true mean value); and (2) total settlements from modified parameters (cautious estimate of the mean value). An unmodified parameter is a term used in this study to refer to values of ground properties that were not subjected to the cautious assessment established by EC7 (CEN, 2004). Instead, best estimate values were used as defined in October 2019 draft. On the other hand, modification of soil parameters involved the estimation of the characteristic values.

As was explained in Chapter 3, the measurements from field and laboratory tests represent only the properties of discrete soil samples. The volume of soil involved in a limit state is much larger and the average soil property within that soil domain will control the possibility of exceeding a limit state (Orr, 2017). The relevant soil property influencing a limit state (e.g., settlements) within a spatial domain of interest is known as “spatial average property” (Tang et al., 1995). An exception to this situation is extremely low or high values of soil properties leading to failure in zones of weak soil. In all the study cases, the selected parameters controlling the settlement of the soil layers were considered as spatial average properties. From a statistical approach, the mean value of the test results is the most probable value of the soil properties since it is not possible to determine the true statistical mean due to the low sample size (Schneider, 1997). Similarly, it is not possible to determine the distribution type followed by the soil property due to a low sample size; then a normal distribution of the soil property is assumed and consequently, the mean is used as the most probable value.

Best estimate approach

According to October 2019 draft, the best estimate of material properties and actions is used to predict soil behavior. Conversely, the characteristic values accounting for sources of uncertainty in the prediction of soil behavior are used for limit states verification. This means that uncertainty is neglected when selecting the best estimate of a soil property, in order to obtain an accurate estimate of the performance of geotechnical structures. If site observations are available, best estimates of performance can be used to compare predicted with observed values of performance. A back-analysis allows adjusting the best estimates of material properties to reproduce the soil behavior described by the observational method. The best estimate term is another addition to EC7 proposed in October 2019 draft. The term is defined in different clauses of the October 2019 draft which are summarized in Table 1.

Table 1. Definitions of the term "best estimate" included in the October 2019 draft of prEN 1997-1.

Chapter/clause	**Definition**
Clause **3.1.3.5** Best estimate value of a ground property	*"Estimate of the most probable value of a ground property".*
Clause **4.3.2** Material and product properties	*"Best-estimate values should be obtained as:* *− the mean or the median of a sample of derived values, whichever it is considered more appropriate;* *− the values used in back-analysis to reproduce the performance of a geotechnical structure known by monitoring".*
Note under clause **4.3.2** Material and product properties	*"Best-estimate values are used for prognosis in contrast to the representative values that are used for SLS verification".*
Chapter **9** Serviceability limit states	*"Best-estimate predictions of performance should use best-estimate values of material properties and actions".* and *"Best-estimate prediction of performance is used for comparison with site observations".*

Predicting soil behavior means selecting the most probable value of ground properties. Hence, considering spatial average soil properties, the best estimate of the true mean (population mean) is the mean of the derived values of the ground property (sample mean). The best estimate value can also be the median, when applicable. Following this definition, the best estimates computed for the estimation of total settlements in this thesis correspond to the arithmetic mean value of the properties obtained from the results of oedometer tests. For each soil layer defined in each study case, the best estimate was calculated.

Since the true mean remains unknown due to the statistical uncertainty introduced by a limited number of samples (Schneider 1997), best estimates might or might not, offer a safety margin. In fact, for design proposes, EC7 considers the characteristic value as the best possible estimate of the true mean value instead of the mean value of the tests (Schneider 1995) because the uncertainty has always to be accounted for. In any case, the best estimate as used in this thesis is less conservative than the characteristic value approach, which accounts for the inherent variability of soil properties. Therefore, it is the base to quantify the safety margin offered by the characteristic value approach.

Characteristic values determination

After the mean of the test results (i.e., the best estimate of the true mean) was calculated per soil layer, the parameters were modified by calculating the characteristic value based on statistical methods. Characteristic values were computed as the cautious estimate of the mean value of the soil parameters using the equation presented in the October 2019 draft (Equation 20). The coefficient k_n in Equation (21) accounts for the uncertainty introduced by the sampling size and it can only be used for cases where more than two tests are available per soil layer. This limitation restricted the applicability of Equation (21) to the soil properties obtained for the Kujala case since there is only one oedometer test performed per soil layer.

Schneider (1997) demonstrated that statistical-based equations for estimation of the characteristic values can be successfully applied in cases with more than 13 samples. With less than 13 samples, the estimation of characteristic values is "low" and "pessimistic". Nevertheless, Schneider (1997) also proposed an equation to determine the characteristic value as a cautious estimate of the mean value with a confidence level of 95%. The equation was calibrated for 13 test results. According to Schneider (1997), the equation can be applied even in cases where there are no test results at all. Likewise, Equation (22) applies to all types of distribution in contrast with the October 2019 draft equation, which is based on the assumption that the soil property values follow a normal or log-normal distribution.

$$X_k = X_{mean}(1 - 0.5\, V_x) \tag{22}$$

The negative symbol (-) in the equation is used in the same way as in October 2019 draft equation. A negative symbol (-) shall be used when conservative values of the soil property are determined to be below the mean value, whereas the positive symbol (+) is used for cases where estimates above the mean value are the most conservative. For instance, a cautious estimate of the mean value of C_c in settlement calculations is a value above the mean of the derived value since it would yield higher – and more conservative – values of settlement than the mean. An exception to this rule is negative soil parameters such as the stress exponent β used in the deformation model by Janbu (1970) in cases where $\beta<0$. In those cases, the positive symbol (+) of Equations (21) and (22) will return conservatively chosen values below the statistical mean.

Equation (22) was used in the Kujala embankment case due to the difficulty to apply a statistical approach. However, for comparison, Schneider's equation usage was extended to all the cases described in Chapter 4. This will allow evaluating the characteristic value approach for cases where only one oedometer test is available per soil layer.

Coefficient of variation of soil parameters V_x

Equations (20) and (21) require an assumed or calculated coefficient of variation of the ground property according to the cases described in Chapter 3.4.1. The October 2019 draft recommends assuming a conservative upper bound of indicative V_x values reported in the literature when little or no data is available at a site. There is only a slight variation in values of V_x when the same soil properties of different sites around the world are compared (Uzielli et al., 2006). Therefore, values of V_x from literature can be used with some confidence when little or no specific-site data is available (Phoon & Kulhaway, 1999b in Uzielli et al., 2006). Moreover, Löfman and Korkiala-Tanttu (2019) observed that values of V_x for compressibility parameters of some selected sediments in Finland are within typical ranges found in the literature. Likewise, they concluded that V_x is not affected by the geological sediment type. The same observation was made by Uzielli et al. (2006). Annex B of the October 2019 draft includes suggested values of V_x to be used in the estimation of the characteristic value of some ground properties. The coefficients of variation of soil parameters for the study cases were selected based on the values reported by Uzielli et al. (2006) and the October 2019 draft. These values are presented in Table 2.

Table 2. Approximate guidelines for selection of V_x assumed.

Soil property	**V_x [%]**
Modulus of deformability[a]	20-70
Compression index C_c [b]	10-37
Compression index C_c [c]	15-24
Swelling index C_s	10-37
Swelling index C_s [c]	11-43
Preconsolidation pressure σ'_p [b]	10-35
Coefficient of consolidation C_v [b]	33-68

[a] October 2019 draft of prEN1997-1
[b] Uzielli et al. (2006)
[c] Löfman and Korkiala-Tanttu (2019)

As it is observed in Table 2, values of coefficient of variation are within different range for different soil parameters, and therefore, the values of V_x for the characteristic values of soil properties were selected accordingly. For instance, the range of V_x values can be considerably wide for modulus of deformability (20 - 70%), but narrower for compression and swelling index (10 - 35%). According to the Annex B of the October 2019 draft, the modulus of deformability refers to the different modulus of deformation whose symbols appear in EN 1997-2 (CEN, 2007). These modulus include the oedometer modulus. The ranges reported in the literature are a reflection of the variability of soil properties that is obtained from laboratory tests on soil samples from different sites and with different quality and sizes. As a result, there will be differences in the reported ranges and even high variability within the ranges themselves.

5.2 Settlement analyses

5.2.1 Layering and location of the measurements

The settlement analyses performed in this thesis consisted of the computation of total settlements from primary consolidation using Novapoint GeoCalc 4.0 software (Civilpoint, 2019). GeoCalc 4.0 calculates the development of pore pressure in primary consolidation based on Terzaghi's one-dimensional consolidation theory. Measurements of total settlement were performed for different calculation points defined along a finite segment of the cross-section of the subsoil beneath the embankments. However, the values of settlement used in this study correspond to the total settlement measured at the centerline of the embankment. Deformation models from tangent modulus and compression index methods were used for all the calculations performed. Soil parameters required in each method were obtained from the results of oedometer tests performed for previous studies. Tangent modulus and compression index methods are described in Chapter 2.4.1 and Chapter 2.4.2, respectively.

The stratification of the subsoil required for the calculations was made using the results of index properties shown in Chapter 4. The soil profile in each case was divided into calculation layers, which were assumed homogeneous. Calculation layers for the Haarajoki case are illustrated in Figure 17. The idealization of the subsoil for Kujala, Murro, and Östersundom cases are presented in Appendix 2. A collection of all the values of soil parameters from oedometer and other test results for each case is presented in Appendix 3.

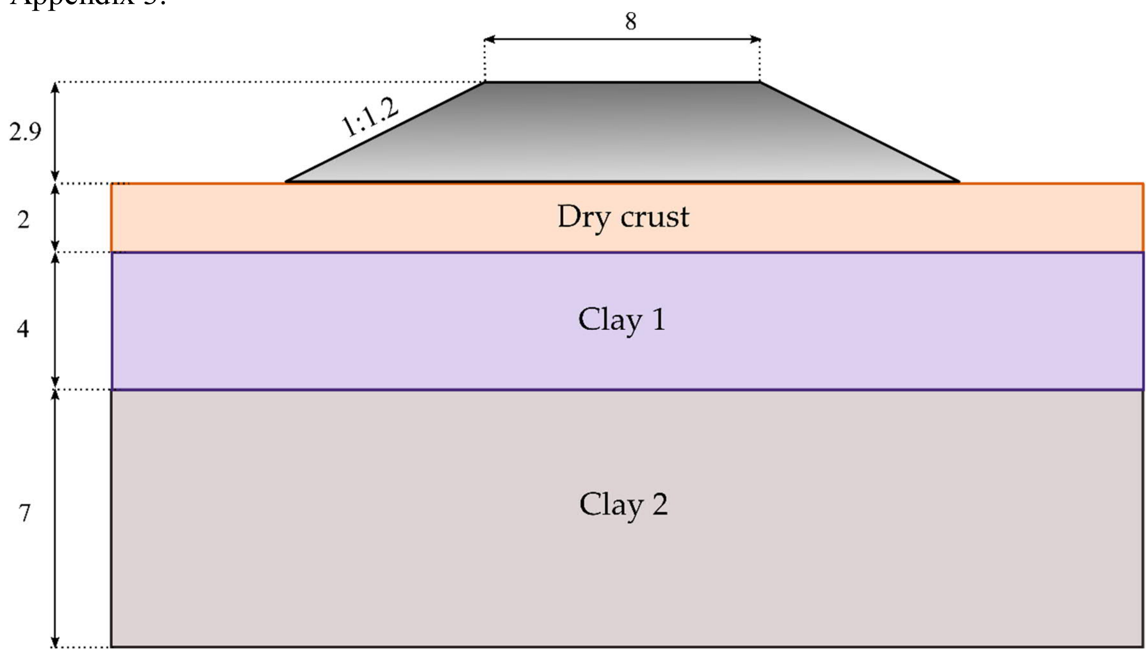


Figure 17. Layer division for the Haarajoki case.

The best estimate and the characteristic values were computed from the soil parameters contained by each layer according to the methodology described in Chapter 5.1. Since V_x values of soil properties in Table 2 are reported as a range within which, the values typically fluctuate, it was necessary to estimate several characteristic values based on different V_x values. For each range of V_x, a characteristic value of the soil properties were calculated using the lower and upper bounds, as well as a value between the bounds. A list of the different settlement analyses performed in this thesis and the exact values of V_x used is presented in Tables 3 and 4 of Chapter 5.2.3.

5.2.2 Tangent Modulus method

The settlements in all the cases were calculated by the tangent modulus method with the GeoCalc 4.0 calculation program (Civilpoint, 2019). GeoCalc's settlement calculation is based on the assumption of one-dimensional consolidation. The vertical stress increment in the program is calculated using the elastic theory, and the excess pore water pressure development is calculated by the finite element method using Terzaghi's theory of one-dimensional consolidation. Terzaghi's theory is presented in Chapter 2.2.

Characteristic values of soil properties were calculated for σ'_p, m, and β parameters in the analyses carried out using the tangent modulus method. Unit weight was left unmodified throughout the different analyses since its variability within a homogeneous soil layer is negligible for Finnish clays (Löfman & Korkiala-Tanttu, 2019). Neither EC7 nor October 2019 draft defines how to select the soil properties influencing a limit state when there are multiple correlated parameters in a calculation model. One alternative is to determine characteristic values for each soil parameter. However, this alternative does not consider the correlation between parameters.

The Ohde-Janbu deformation model (Janbu 1970) defines the modulus of compressibility (*M)* that is a stress-dependent parameter as shown in Equation (14). According to Table 2, typical values of V_x for this modulus vary between 20 – 70%, considering that the oedometer modulus is included in the group of the modulus of deformation that appear in in EN 1997-2 (CEN, 2007). Specific values for m and β parameters were not found in the literature. Therefore, the same range of values of V_x for M was used for m and β considering the relationship among the parameters and the high degree of variability that is accounted for within this range.

The value of the stress exponent in Finnish clays typically varies between $\beta = 0 \ldots -1$. Positive values are also possible. The modulus numbers m and the stress exponents β were determined using a curve fitting method developed at Helsinki University of Technology. In this method, the best fit is obtained for the stress-strain curves from oedometer tests. On the other hand, the input values of σ'_p for overconsolidated layers were defined either as a constant value for the entire layer or in terms of a pre-overburden pressure (POP). The decision was based on the distribution of the preconsolidation pressure with depth. A calculation example of characteristic values for the tangent modulus method is presented in Appendix 4. A summary of all the input parameters used in each study case for the tangent modulus method is found in Appendix 5. The summary includes the best estimates calculated for each soil layer and the characteristic values of soil parameters.

Modulus number adjustment

A common error when calculating settlements with tangent modulus method is the use of values of m and β without considering the stress range from which they are determined. During the calculation of characteristic values, smaller preconsolidation pressures than the measured from oedometer were obtained. Thus, the input parameters for the tangent modulus method were used for preconsolidation pressures different from the oedometer preconsolidation pressure. This leads to a stress-strain behavior different from the observed for the entire layer (Länsivaara 2003). However, the oedometer stress-strain curve is unique for the entire clay layer and the preconsolidation pressure is strain-rate dependent. Therefore, a correction of values of m needs to be performed to connect these values with the preconsolidation pressure from which, it has been determined. Länsivaara (1995) proposed a method to modify m and avoid the calculation error described above (Equation 23):

$$m_{calculation} = m_{test} \cdot \left(\frac{\sigma'_{cv\ test}}{\sigma'_{cv\ calculation}} \right)^{-\alpha} \tag{23}$$

Where $m_{calculation}$ is the modulus number to be used in settlement calculations
m_{test} is the modulus number obtained from oedometer tests
$\sigma'_{cv\ test}$ is the preconsolidation pressure obtained from oedometer tests

$\sigma'_{cv\ calculation}$ is the preconsolidation pressure used in the calculation
α is the stress exponent (*β*).

Equation (23) is already integrated into GeoCalc's settlement calculation program. The user, however, has to define the preconsolidation pressure obtained from oedometer tests ($\sigma'_{cv\ test}$). For the settlement analyses performed in this thesis, $\sigma'_{cv\ test}$ corresponded to the highest value of σ'_p from oedometer tests in each layer. These corrections allowed having settlements values similar to the values obtained with the compression index method and closer to the predicted and measured settlements values. Before performing the correction, the values of settlement were significantly higher than the values obtained from the compression index method. Thus, higher structuration effects were avoided for smaller preconsolidation pressures with this correction.

This stress-strain behavior deviation is especially relevant in soft clays experiencing structuration effects. Negative β values are observed in soft clays with high destructuration, and with higher negative β values the correction made with Equation (23) becomes more significant. In all the selected study cases except in the Kujala case, negative β values were observed. Thus, it was necessary to apply the correction for *m* parameter to reduce the calculation error effects on the compression of the layers.

Another calculation error when applying the tangent modulus method is related to high unrealistic vertical deformation values of overconsolidated dry crust layers. Low effective vertical stress is found near the ground surface for dry crust layers. In consequence, the values of *M* will be also small due to the stress-dependency of *M*. This will result in unlikely high vertical deformations of the dry crust, which normally exhibits low compressibility. A solution for this error is to use a stress exponent for the overconsolidated range β_2=1. In this way, the modulus of compressibility will be constant (i.e. independent of stress).

Graphical representation of parameters modification

When modifying soil parameters *m, β* and σ'_p, different stress-strain plots can be obtained per layer. Thus, it is possible to illustrate the effect of applying characteristic values to settlement calculations through modified stress-strain curves. Figure 18 shows the effect of modifying *m, β* and σ'_p on the stress-strain behavior for the first clay layer (see: Figure 17) in the Haarajoki case. The different curves were obtained from (1) best estimates of *m, β,* and σ'_p; (2) characteristic values of *m, β,* and σ'_p using Schneider's equation and a $V_x(m, \beta)$=0.20 and $V_x(\sigma'_p)$=0.10 with modulus number adjustment; (3) characteristic values of *m, β,* and σ'_p using Schneider's equation and a $V_x(m, \beta)$=0.70 and $V_x(\sigma'_p)$=0.35 with modulus number adjustment; (4) characteristic values of *m, β,* and σ'_p using Schneider's equation and a $V_x(m, \beta)$=0.20 and $V_x(\sigma'_p)$=0.10 without modulus number adjustment; (5) characteristic value of σ'_p using Schneider's equation and a $V_x(\sigma'_p)$=0.35, with modulus number adjustment. Figure 18 shows how modifying *m, β,* and σ'_p results in larger strains for the same stress levels. Likewise, the deviation of the stress-strain behavior produced by the addition of a safety margin is more critical when the modulus number adjustment is not carried out.

In the Haarajoki case, the vertical stress increment produced by the embankment load is slightly larger than the preconsolidation stress. Hence, the reduction of the preconsolidation pressure carried out when calculating its characteristic value means that more of the additional stresses from the embankment weight will be in the compression range (i.e., normally consolidated area of the curve) when compared to the best estimates curve (1). This led to considering the effect on the stress-strain when only σ'_p is modified. In Figure 18, is evident from the curve (5) that σ'_p modification has an important contribution to the stress-strain behavior deviation caused by the addition of a safety margin. This is an important consideration when deciding the soil parameters to be modified.

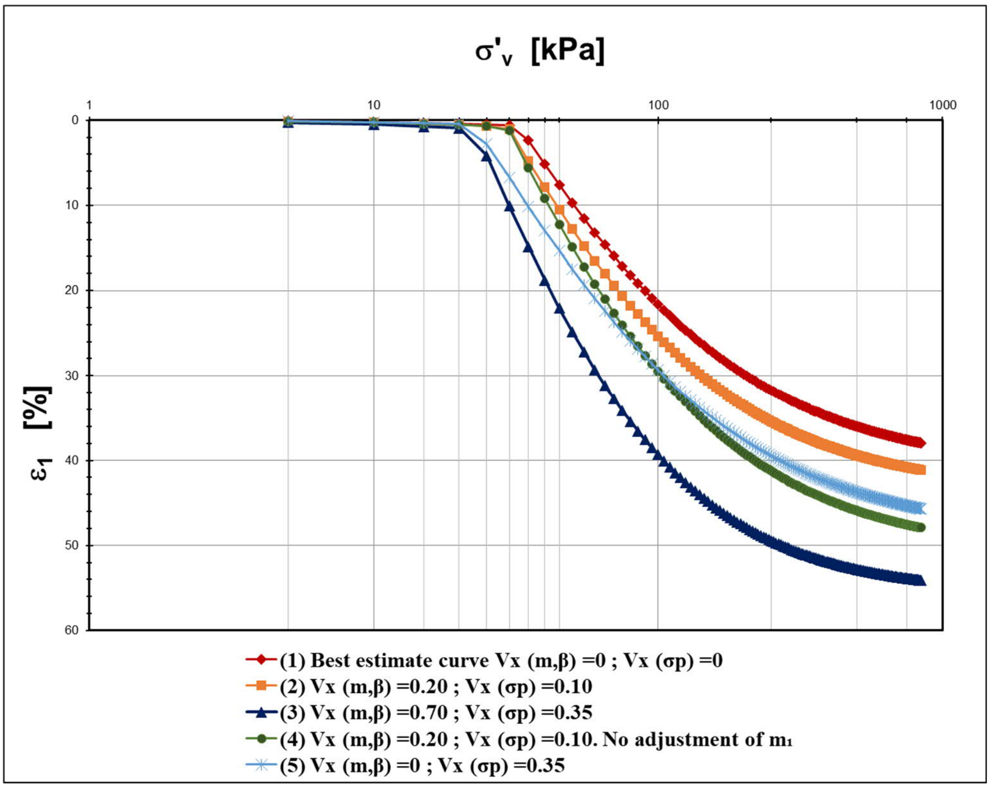


Figure 18. Stress-strain plots resulting from modified and unmodified soil parameters used for the tangent modulus method. Curves correspond to soil parameters of the first clay layer in the Haarajoki case.

5.2.3 Compression index method

In addition to the tangent modulus method, settlement analyses were replicated using the compression index method with the GeoCalc 4.0 calculation program (Civilpoint, 2019). The same assumptions made in the tangent modulus method are applied in the compression index method. However, the input parameters are different. In GeoCalc 4.0, the input parameters are the compression index (C_c), swelling index (C_s), initial void ratio (e_o) and σ'_p. C_c is the deformation parameter in the normally consolidated part while C_s accounts for deformations in the overconsolidated range. The input parameters in the compression index method were obtained from oedometer tests.

Characteristic values estimation was applied to values of C_c, C_s, and σ'_p, while e_o parameter remained as best estimates throughout the analyses. Hence, e_o was not treated as an intrinsic compressibility parameter but rather as an input parameter whose variability-related uncertainty can be neglected. Values of σ'_p for each layer in all the studies cases were defined as described in Chapter 5.2.2. A summary of all the input parameters used in each study case for the compression index method is found in Appendix 3. The characteristic values of these input parameters for different coefficients of variation are found in Appendix 6.

5.2.4 Settlement analyses summary

The two main settlement analyses performed for each case in this thesis: (1) total settlement from unmodified parameters, and (2) total settlement from modified parameters. In turn, both analyses are divided into a set of sub-analyses based on the settlement calculation method, the statistically-based equation to estimate the characteristic values of soil parameters, and the values of V_x used in those equations. Tables 3 and 4 enlist the total settlement analyses performed along with the considerations taken in each analysis. Analysis 6 in Table 4 corresponds to a calculation of total settlement performed for Östersundom case, in which, the coefficient k_n of Equation (21) is estimated for a number of samples *n=2*. Two samples are the minimum number required for the applicability of Equation (21). The aim of analysis 6 is to evaluate the additional conservatism provided by the uncertainty accounting for the small size of a sample, which is quantified by the coefficient k_n (sample uncertainty).

Table 3. Settlement analysis from unmodified parameters (Best estimates)

Sub analyses	Equation	Parameter (V_x)	Method	Cases
Analysis 1a	NA	β (V_x=0), m(V_x=0) σ_c(V_x=0%)	Tangent Modulus	All cases
Analysis 1b	NA	C_c(V_x=0), C_s(V_x=0) σ_c(V_x=0	Compression index	All cases

Table 4. Settlement analysis from modified parameters (characteristic values)

Sub analyses	Equation	Parameter (V_x)	Method	Cases
Analysis 2a	2019 October draft	β (V_x=0.20) m(V_x=0.20) σ_c(V_x=0.10)	Tangent Modulus	Haarajoki Murro Östersundom
Analysis 2b	2019 October draft	β (V_x=0.40) m(V_x=0.40) σ_c(V_x=0.23)	Tangent Modulus	Haarajoki Murro Östersundom
Analysis 2c	2019 October draft	β (V_x=0.70) m(V_x=0.70) σ_c(V_x=0.35)	Tangent Modulus	Haarajoki Murro Östersundom
Analysis 3a	2019 October draft	C_c(V_x=0.10), C_s(V_x=0.10) σ_c(V_x=0.10)	Compression Index	Haarajoki Murro Östersundom
Analysis 3b	2019 October draft	C_c(V_x=0.24), C_s(V_x=0.24) σ_c(V_x=0.23)	Compression Index	Haarajoki Murro Östersundom
Analysis 3c	2019 October draft	C_c(V_x=0.37), C_s(V_x=0.37) σ_c(V_x=0.35)	Compression Index	Haarajoki Murro Östersundom

Table 4. Settlement analysis from modified parameters (characteristic values)

Sub analyses	Equation	Parameter (V_x)	Method	Cases
Analysis 4a	Schneider	β (V_x=0.20) m(V_x=0.20) σ_c(V_x=0.10)	Tangent Modulus	All cases
Analysis 4b	Schneider	β (V_x=0.40) m(V_x=0.40) σ_c(V_x=0.23)	Tangent Modulus	All cases
Analysis 4c	Schneider	β (V_x=0.70) m(V_x=0.70) σ_c(V_x=0.35)	Tangent Modulus	All cases
Analysis 5a	Schneider	C_c(V_x=0.10), C_s(V_x=0.10) σ_c(V_x=0.10)	Compression Index	All cases
Analysis 5b	Schneider	C_c(V_x=0.24), C_s(V_x=0.24) σ_c(V_x=0.23)	Compression Index	All cases
Analysis 5c	Schneider	C_c(V_x=0.37), C_s(V_x=0.37) σ_c(V_x=0.35)	Compression Index	All cases
Analysis 6a	2019 October draft	β (V_x=0.20) m(V_x=0.20) σ_c(V_x=0.10)	Tangent Modulus	Östersundom
Analysis 6b	2019 October draft	β (V_x=0.40) m(V_x=0.40) σ_c(V_x=0.23)	Tangent Modulus	Östersundom
Analysis 6c	2019 October draft	β (V_x=0.70) m(V_x=0.70) σ_c(V_x=0.35)	Tangent Modulus	Östersundom

Each of the analyses in Table 4 was performed using characteristic values as the cautious estimates of the mean value of the soil properties. In addition, the analyses from 2a to 5c were replicated using a favorable estimate of the mean value of the soil property. A favorable estimate of the mean value corresponds to a value yielding less conservative estimates of settlements than the mean of the derived values.

5.3 Time-settlement analysis

A time-settlement analysis for soil consolidation in the Haarajoki case was carried out in GeoCalc 4.0 (Civilpoint, 2019) to evaluate the effect of modifying the coefficients of consolidation in the degree of consolidation. The assessment was performed at different times after the embankment construction (after 2 and 3.5 years). The modification of soil parameters was based on the cautious assessment of EC7, using the percentage range of coefficient of variations presented in Table 2. The best estimate calculation was used as previously described to select the unmodified *Cv* parameters.

GeoCalc's time-settlement calculation is based on the one-dimensional consolidation theory of Terzaghi (Terzaghi & Peck, 1961). The input values of *Cv* have to be given for overconsolidated and normally consolidated ranges. The values of *Cv* for the overconsolidated range are approximately 10 times larger than the normally consolidated part in Finnish soft clays. Coefficients of consolidation for each layer were defined from oedometer tests performed as presented in chapter 2. However, the oedometer-based *Cv* values yielded considerable conservative results for the degree of consolidation assessed at 2 and 3.5 years. These *Cv* values are reported in Appendix 3. Therefore, it was necessary to apply alternative methods to obtain a more realistic *Cv* for the soft clay of the Haraajoki case. The method used consisted of a back-analysis of *Cv* values based on the observed settlement values reported by Länsivaara (2001) for the Haraajoki embankment. The back analysis was performed by multiplying the values of *Cv* per layer by an increasing factor (starting from 2) until obtaining an approximate value of settlement. Further adjustment was performed later on to the individual factors to make the result more approximate to the observed values.

In addition to the observed settlement values reported for Haraajoki embankment, the study by Länsivaara (2001) makes also settlement predictions over 20 years after construction for different observation times and using different prediction methods. Figure 19 shows the observed values of total settlement over 3.5 years and the predicted values for 20 years after construction based on the longest observational time, when different methods are used. Settlements of Figure 19 corresponds to the values measured at the centerline of the embankment for the portion of soil deposit without vertical drains.

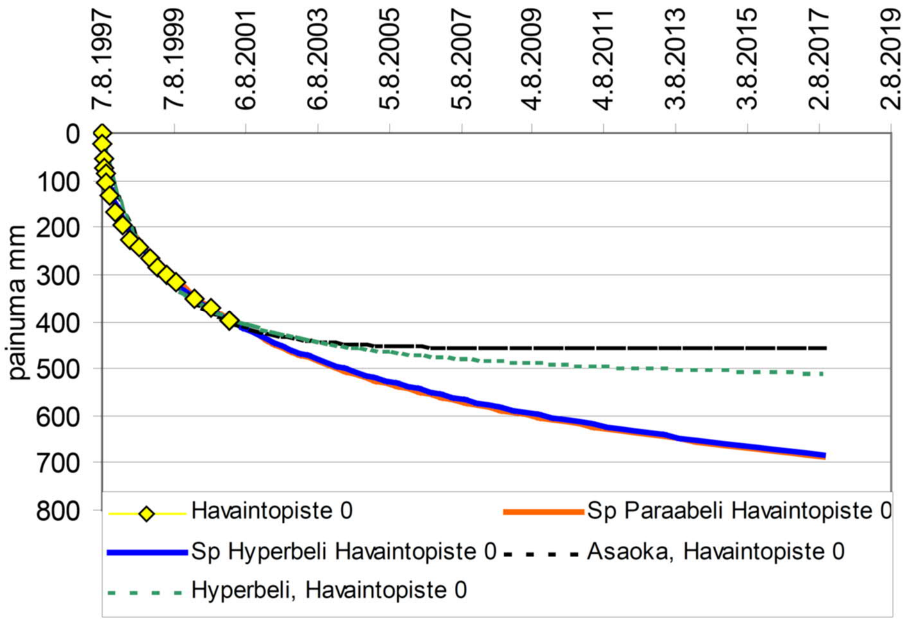


Figure 19. Observed and predicted settlements values reported by Länsivaara (2001) for the Haarajoki embankment.

According to the study by Länsivaara (2001), the application of the settlement potential method ("*Sp Paraabeli"* and "*Sp Hyperpeli*") resulted in better predictions than the other methods evaluated in figure 19. As it is shown in Figure 19, a settlement of approximately 400 mm was observed at 3.5 years, whereas a value of 686 mm is predicted at 20 years based on the potential method. These were the reference values used to correct the values of C_v provided by oedometer tests in the Haarajoki case. However, the adjustment was done only for the period that is being considered in Figure 19 and therefore, the total settlement from primary consolidation does not necessarily have to be the same.

Corrected values of C_v were treated as best estimates or unmodified values, which were modified to obtain characteristic values and using the range of coefficient of variation presented in chapter 5 (33 - 68%). Characteristic values were only estimated for C_v, leaving the rest of the parameters used in each time-settlement analysis as unmodified parameters (best estimates). Likewise, considering the lack of laboratory data, only Schneider's equation was applied in the estimation of the characteristic values of C_v. The equation was applied in such a way that the characteristic value of C_v becomes smaller for higher coefficients of variation, resulting in a slower consolidation process for the most conservative analyses.

6 Results and discussion

This chapter presents the results of the different settlement calculations carried out using characteristic values (i.e., characteristic values) and best estimate values of soil parameters in each studied case, based on the definitions provided by European standard Eurocode 7 (CEN, 2004) and October 2019 draft. In addition, the chapter discusses how the selection of these parameters affects the settlement calculations needed in SLS verification and the necessity of implementing partial factors different from the unity in settlement calculations of fine-grained soils to provide sufficient safety margin.

6.1 Primary consolidation analyses

Settlement vs. Coefficient of variation

Total settlement resulting from primary consolidation was obtained for each study case according to the procedure described in chapter 5, with characteristic values defined as the cautious (unfavorable) estimate of the mean value and a favorable estimate of the mean value. These values are summarized in Figure 6.1, where the total settlements calculated by using modified (characteristic values) and unmodified (best estimate) parameters are plotted against the different coefficients of variation that were used for the characteristic values of the compressibility parameters. Results are separated by the characteristic values calculated with October 2019 draft and Schneider's equations and two plots were obtained for each case; one corresponding to the tangent modulus method and the other one to the compression index method.

As it is observed in the plots, settlements calculations are sensitive for higher values of coefficients of variation applied to the respective soil parameters, especially in the case of settlements parameters m and β, where a wider range for the coefficient of variation was used (20-70%) with respect to the compression and swelling index (10-37%). Likewise, the use of October 2019 draft equation yielded more conservative results than the Schneider's equation. However, this is only true for a certain number of test results, as it was observed that in cases with a relatively high number of test results per layer, October 2019 draft equation is less conservative than Schneider's equation when the parameters were evaluated individually. For instance, in the Östersundom case, the characteristic value of the preconsolidation pressure of the clay layer with the highest number of test results (12) was slightly higher than the value obtained with Schneider′s equation. This is because October 2019 draft equation uses a coefficient K_n that depends on the number of samples and that accounts for the uncertainty produced by the number of available tests per layer, whereas Schneider's equation is applicable in the same form for cases with less than 13 available tests results (Orr 2017). Thus, for a limited number of test results, October 2019 draft equation gets more conservative while the use of Schneider's equation gives less overestimated settlement values.

The values obtained were compared to the observed and predicted values of primary consolidation that have been reported in studies on the investigated embankment cases. Values are reported in Chapter 4. Although the settlement values obtained in this study slightly differ from predicted and observed values in some cases, they are on the same scale of results for each case. It is worth noticing that the observation periods in the study cases correspond to the first years after construction and therefore, it is probable that the observed settlement is not the same as the real final settlement of primary consolidation.

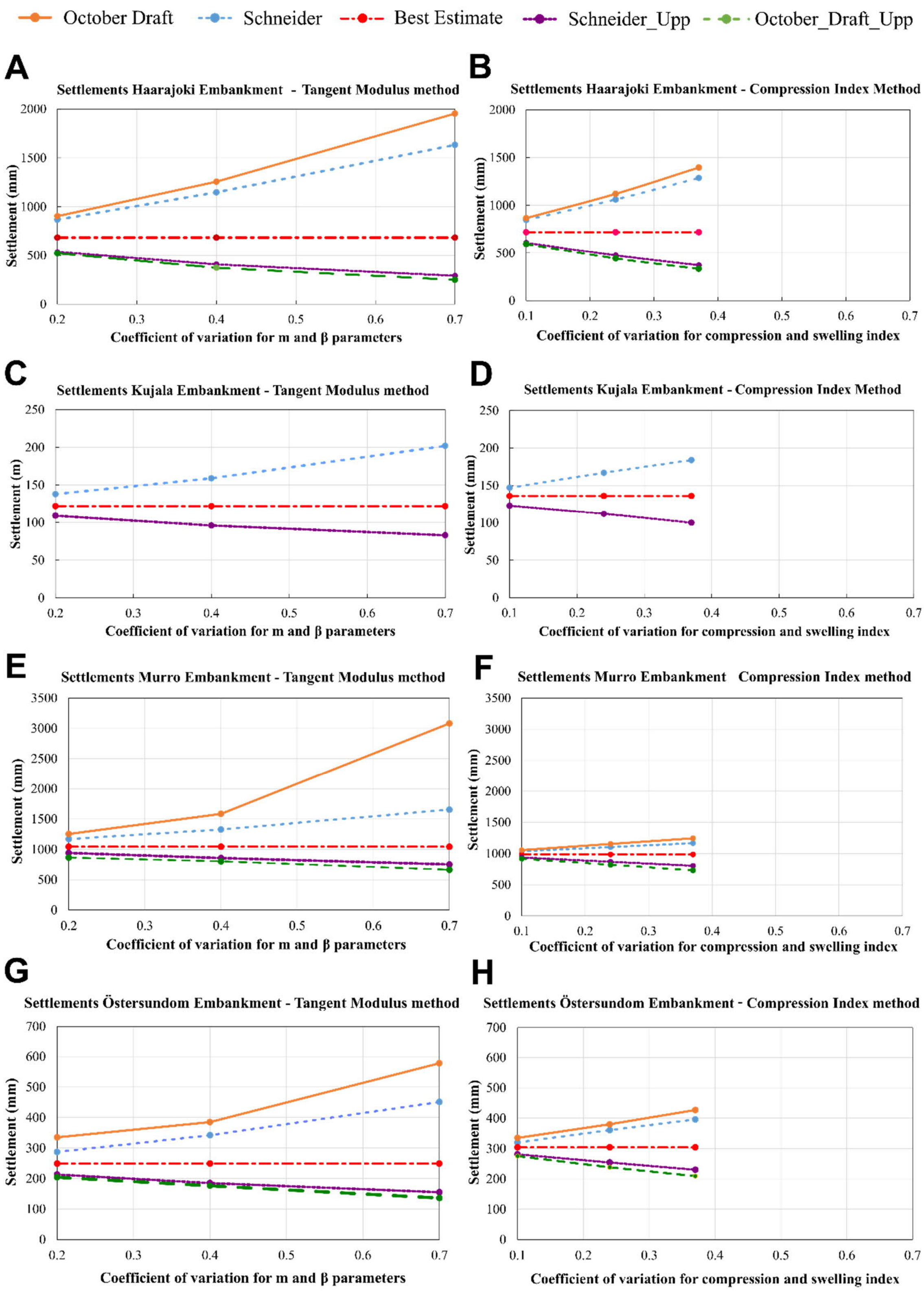


Figure 20. Total settlement as function of the coefficient of variation used in the estimation of the characteristic values of settlement parameters for Haarajoki test embankment (A) and (B), Kujala embankment (C) and (D), Murro test embankment (E) and (F) and Östersundom test embankment (G) and (H). Values corresponding to the series Schenider_Upp and October_Draft_upp are calculated from favorable estimates of the mean value of soil parameters.

The plots in Figure 20 correspond to the settlement analyses enlisted in Tables 3 and 4 from analyses 1a to 5c. Values of V_x for σ'_p corresponds to 0.10, 0.23 and 0.35 when V_x in the plots for m and β are 0.20, 0.40, and 0.70 respectively.

Figure 21 shows the results from analysis 6, where settlements were calculated from characteristic values of parameters σ_c, *m,* and β using October 2019 draft and a coefficient of k_n corresponding to a number of samples *n=2*. The number of tests per layer in Östersundom case is unusually high; some layers have up to 12 test results. This allows evaluating the conservatism provided by the characteristic values approach as a function of the coefficient of variation for cases meeting the minimum requirement for the application of October 2019 draft (*n*=2), which could be a typical case in SLS analyses in geotechnical projects.

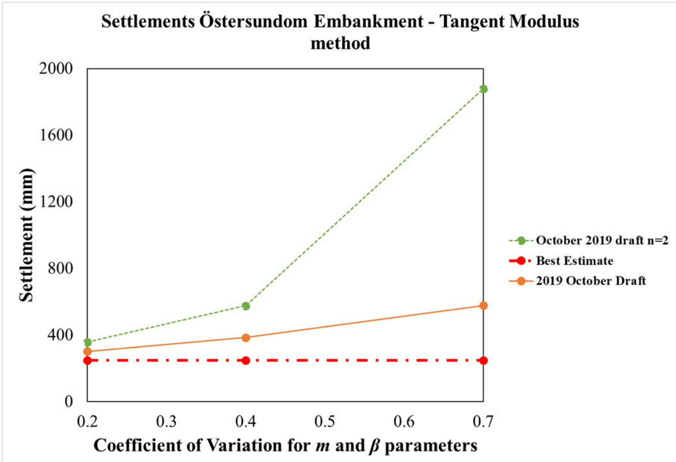


Figure 21. Results from analysis 6. Östersundom.

Figure 21 shows large conservative settlements obtained from the upper bound of the coefficient of variation applied to m and β. However, for the lowest value of V_x within the selected range, results do not differ too much from the values obtained when k_n corresponds to the actual number of tests. Coefficient of k_n accounts for the sampling uncertainty introduced by the number of samples. From Figure 21, it is observed that larger sampling uncertainties are affected more severely with increasing inherent variability of the soil parameters V_x.

Effects on cumulative settlement with depth

Another relevant comparison when evaluating the effects of modifying ground properties in settlements calculations is related to the cumulative settlement with depth, whose change in magnitude reflects the compressibility exhibited by every layer of the soil deposit. Naturally, the response of the soil produced by the effect of the embankment load varied in every layer and due to the non-linearity of the stress-strain relationship, the modification of soil parameters based on the characteristic value approach might affect the response of each soil layer in a different way, and therefore this effect was evaluated in each case. Figures 22 (A) and (B) show the effect of parameter modification on the compressibility of each layer in Haarajoki case, when total settlements of the centerline are calculated using tangent modulus (TM) and compression index method (CI) and different coefficients of variation V_x are used in characteristic values estimation. The equation used to modify the values of Figure 22 corresponds to the one provided in the October 2019 draft, which returned more conservative values of settlement than Schneider's equation.

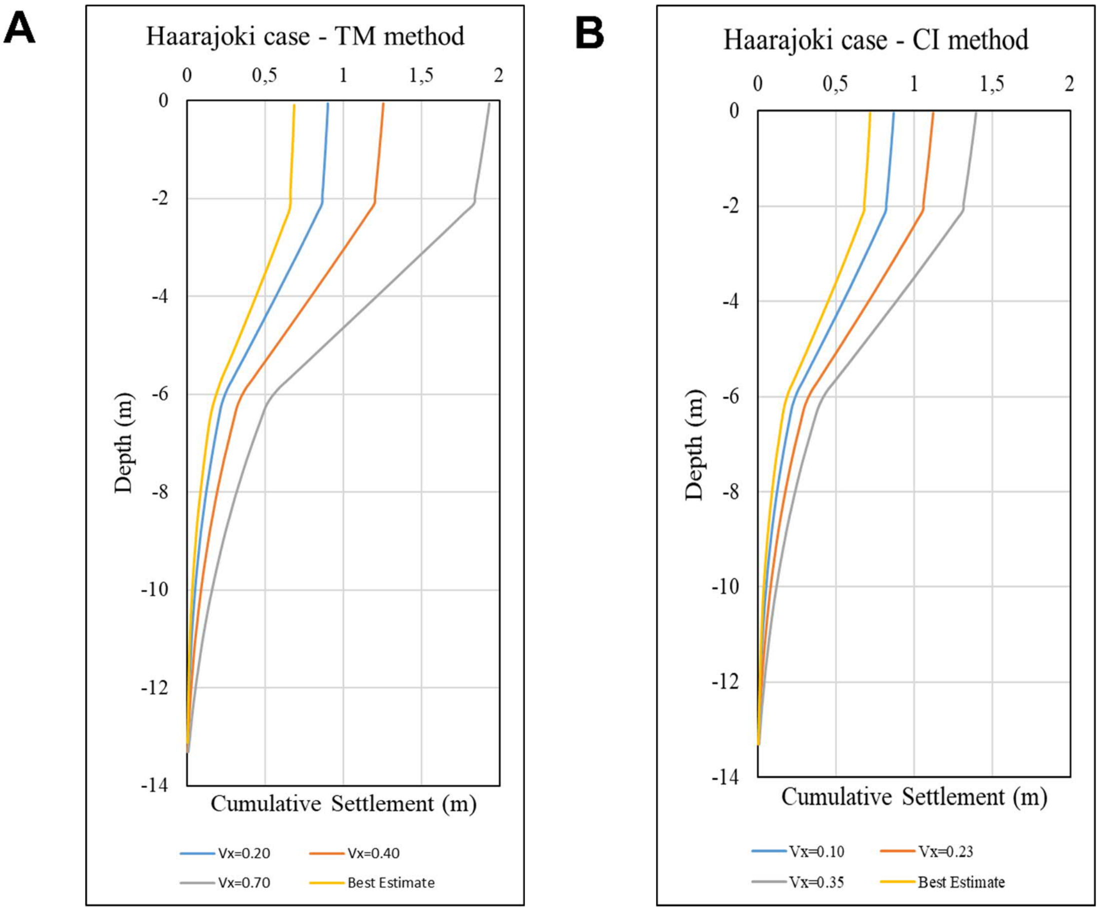


Figure 22. Cumulative settlement with depth in Haarajoki case for tangent modulus method (A) and compression index method (B). Settlements calculated at the centerline of the embankment with characteristic values according to the October 2019 draft.

In Figure 22 (A) and (B) is observed that the amount of compression in each soil layer with respect to the other layers remains consistent with the modification of the soil parameters after using the characteristic value approach. Thus, the soil layer between 2 to 6 m depth continues to be the most compressible layer throughout the analyses. The dry crust layer on the other hand, does not experiment any significant change in its relative compressibility. Likewise, Figure 22 shows the sensitivity of settlement calculations produced by the modification of the m and β parameters when compared to compression and swelling index (since the applied values of coefficient of variation were greater).

Summary and comparison between analyses with modified and unmodified parameters

The differences in soil conditions among the study cases led to different scales of settlement that are not comparable unless the difference between modified and unmodified parameters is quantified by means of a ratio of total settlement calculated from modified parameters –characteristic values- to the total settlement from unmodified parameters –best estimates-. Table 5 and Table 6 provide these ratios for all the study cases when the characteristic values are estimated for the parameters used in tangent modulus and compression index methods, respectively. The results in both tables are separated by the coefficient of variation used for the estimation of the characteristic values of soil parameters and by the equation used. In figure 23 the ratios are depicted in graphical form.

Table 5. Ratio of total settlement from modified to total settlements from unmodified parameters in each study case. Tangent modulus method. V_x values correspond to the ones used for compressibility parameters. See Table 4 for the corresponding values of V_x used for σ'_p.

Case	Best estimate (mm)	Observed values (mm)	Ratio modified to unmodified Schneider´s equation			Ratio modified to unmodified October 2019 draft		
			V_x=0.20	V_x=0.40	V_x=0.70	V_x=0.20	V_x=0.40	V_x=0.70
Haarajoki	685	400[a] (3.5 yr.)	1.3	1.7	2.4	1.3	1.9	2.9
Kujala	122	65-115[b] (1.5 yr.)	1.1	1.3	1.7	-	-	-
Murro	1044	798[c] (8 yr.)	1.2	1.3	1.6	1.2	1.5	3.0
Östersundom	249	130[d] (1.5 yr.)	1.2	1.4	1.8	1.3	1.5	2.3

[a]Länsivaara (2001, p.40-41)
[b]Löfman and Korkiala-Tanttu (2020)
[c]Koskinen et al. (2002)
[d]Köylijärvi (2015)

Table 6. Ratio of total settlement from modified to total settlements from unmodified parameters in each study case. Compression index method.

Case	Best estimate (mm)	Observed values (mm)	Ratio modified to unmodified Schneider´s equation			Ratio modified to unmodified October 2019 draft		
			V_x=0.10	V_x=0.24	V_x=0.37	V_x=0.10	V_x=0.24	V_x=0.37
Haarajoki	719	400 (3.5 yr.)	1.2	1.5	1.8	1.2	1.6	2.0
Kujala	136	65-115 (1.5 yr.)	1.1	1.2	1.4	-	-	-
Murro	986	798 (8 yr.)	1.1	1.1	1.2	1.1	1.2	1.3
Östersundom	304	130 (1.5 yr.)	1.1	1.2	1.3	1.1	1.3	1.4

The ratios values of Tables 5 and 6 are an indication of the safety margin provided by the characteristic values when the mean value of the samples (i.e., best estimate) is used as a basis. In the case of the investigated embankments is close to one (1) when the lowest coefficients of variation are being considered in both settlement calculation methods. However, for a higher coefficient of variation, the ratio values are less uniform among the cases and they can reach values over two in the case of tangent modulus method, meaning that the characteristic values can be twice as big as the best estimate. Likewise, the usage of Schneider's equation in comparison with the October 2019 draft equation has a more remarked effect in the ratios when the tangent modulus method is applied. More critical results are expected for the tangent modulus method considering the wider range of V_x used for soil parameters and the accountability of the method for the nonlinearities of the stress-strain relationship of soft Finnish clays. The October 2019 draft recommends selecting a conservative upper estimate of V_X, but the ratios of modified to unmodified obtained for these cases are quite large.

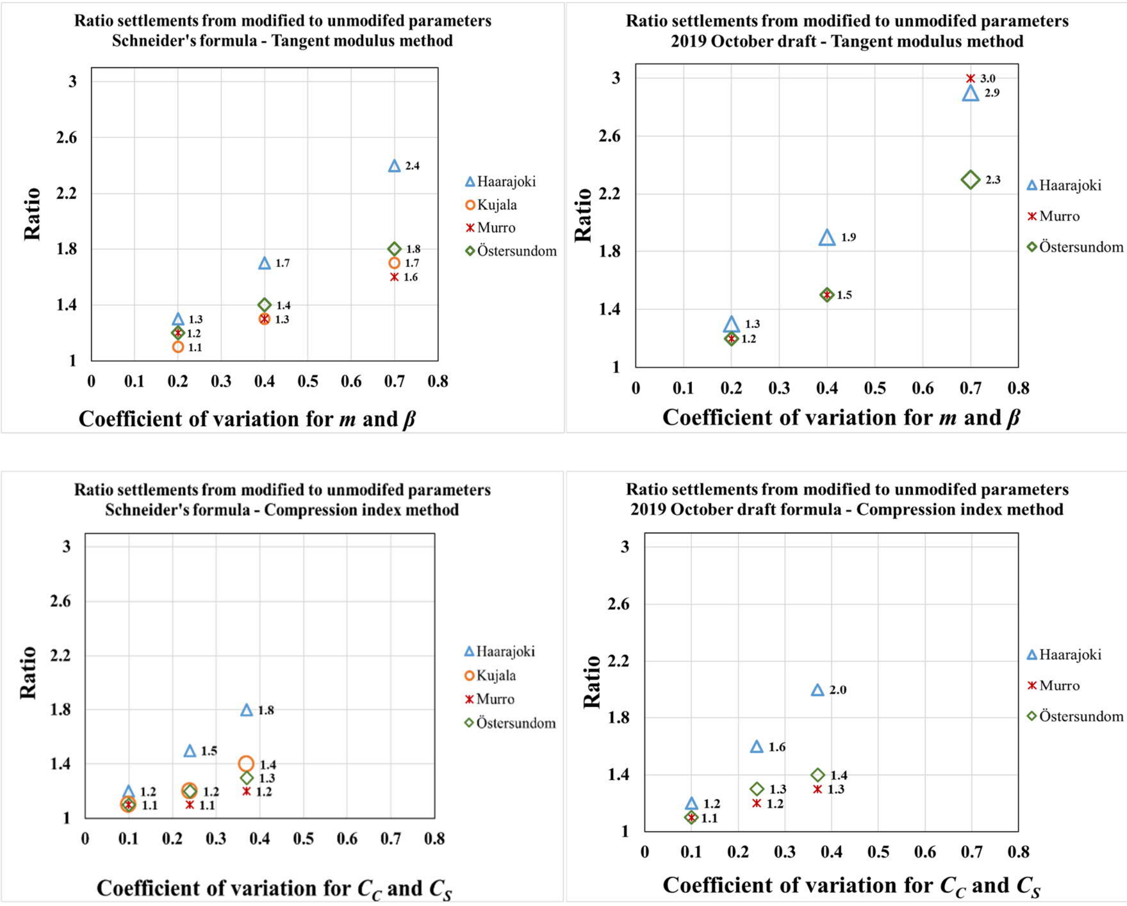


Figure 23. Ratio of total settlement from modified to total settlements from unmodified parameters for all the study cases.

From Figure 23 it is observed that the ratios in the Haarajoki case differ from the ratios of the rest of the cases. The difference becomes more significant with higher values of coefficient of variation. This deviation in the ratio of total settlement from modified to unmodified might be a reflection of the strong nonlinearities of the compression range of the clay layers in the Haarajoki site. The large negative values of β are evidence of these nonlinearities. Likewise, negative stress exponents are connected to the destructuration of soft clays (Länsivaara 2003). The average value of β throughout the soft clay deposit is -0.82, while in the other cases the average of the deposits varies between -0.20 to -0.32. When modifying the β values for characteristic values estimation, the negative values of β become larger, making the nonlinearities and the structuration effects more severe.

Similarly, Murro case presents with respect to the Kujala and Östersundom cases, a considerable deviation of the ratio estimated for tangent modulus method when V_x of m and β is the highest (70%), and when the October 2019 draft equation is applied. The reason behind this difference is connected to the high conservatism provided by the coefficient K_n for high values of V_x, as explained earlier. In two of the five clay layers resulting from the subsoil division in Murro case, there are only two sets of test results (n=2). When n=2, which is the minimum value of test results required for the application of the October 2019 equation, K_n has a value of 1.16. If a V_x of 0.70 is considered, the mean value of the soil parameter is reduced or increased by nearly 80%. This considerable modification of the m and β parameters resulted in a large value of settlement. Moreover, the large conservatism of settlement calculations in Murro case when using V_x of 0.70 for m and β led to an unrealistic value of settlement (3081 mm) above the height of the embankment (2000 mm). This was also the case for Östersundom embankment in analysis 6. In these cases, it would be necessary to modify the embankment load since part of the embankment would settle below the groundwater level.

The ratios in Tables 5 and 6 are comparable with the total factor of safety method commonly applied to stability problems, which is the ratio of the ultimate resistance to the applied loads or loads effects. Terzaghi and Peck (1948) provide customary ranges of total factors of safety to be applied to normal loads and service conditions. For earthworks such as embankments (i.e. embankments), the total factor of safety varies between 1.3 and 1.5. Therefore, ratios in Figure 23 above these values are already providing a large conservatism. Moreover, the total factors of safety presented by Terzaghi and Peck (1948) are used for stability analyses and therefore, they are not fully comparable with serviceability limit states since failure due to instability might pose a life-threatening situation. The difference between ultimate limit state and serviceability limit state is reflected in the design approaches DA defined in EC7 which are divided into consequence classes. However, the total factor of safety serves as a basis for comparison considering that it provides the ratio of soil resistance to applied load or applied load effects as a way to quantify the certainty of the design. Similarly, the ratios in Figure 23 provide the margin of certainty obtained with characteristic values.

Meyerhof (1995) performed a probabilistic analysis to estimate partial factors for different properties with a reliability of 90%. The analyses aimed to compare the results with partial factors given by Eurocode (CEN, 1993) to the SLS design approach. For C_c, partial factors of 1.5 – 2 were estimated. These values were obtained with a coefficient of variation in the range of 25 – 40%. According to Meyerhof (1995), a partial factor of 1.0 for C_c in deformation calculations are suitable for the estimation of allowable movements of earth structures considering that partial factors are applied to characteristic values of soil properties, which are selected as cautious estimates of mean values. Moreover, Meyerhof (1995) states that "*the lower ranges of the coefficient of variation are likely to govern the resistance of in situ properties of large soil masses affecting the stability of earth structures and foundations on any one site in practice*". For these reasons, the author concludes that the partial factor unity given by Eurocode (CEN, 1993) is supported by his comparative results.

This thesis focused on the safety margin provided by the characteristic values in total settlement calculations. However, differential settlements in serviceability limit state verification are also often required. In fact, they might be more significant and decisive than total settlements. Inherent spatial variability of the soil is a major reason for differential settlements to occur. The magnitude of differential settlements is commonly estimated based on empirical correlations between observed total and differential settlements. In these correlations, variance reduction accounting for spatial averaging is not taken into account (Schneider et al., 2015), which can lead to a very conservative values of differential settlements. Hence, a probabilistic approach could be applied to determine the empirical factors applied to the average total settlement for the determination of differential settlements.

6.2 Time-settlement analysis

As it was explained in Chapter 5.3, the coefficients of consolidation obtained through oedometer tests for the Haarajoki test embankment are considerably conservative. Therefore, the resulting time-settlement analysis gives an unrealistic degree of consolidation when compared to observed values reported in Länsivaara (2001) for the first 3.5 years after the embankment construction. The values of C_v were adjusted to obtain a degree of consolidation near the reported for the longest observational time (3.5 years). Table 7 shows the original mean values of C_v obtained from oedometer tests and the calibrated values from the back-analysis.

Table 7. Mean values of C_v from oedometer tests and best-estimate from observed values. Haarajoki case.

Layer	Oedometer test values [m^2/yr]		Back-analyzed values [m^2/yr]	
	C_v OC range	C_v NC range	C_v OC range	C_v NC range
Dry-Crust	9.53	1.17	160	16
Clay 1	0.56	0.06	8	0.8
Clay 2	1.13	0.13	16	1.6

Figure 24 shows the results from the time-settlement analysis performed using best estimates of values of C_v in comparison with the analysis from mean values of C_v obtained from oedometer tests.

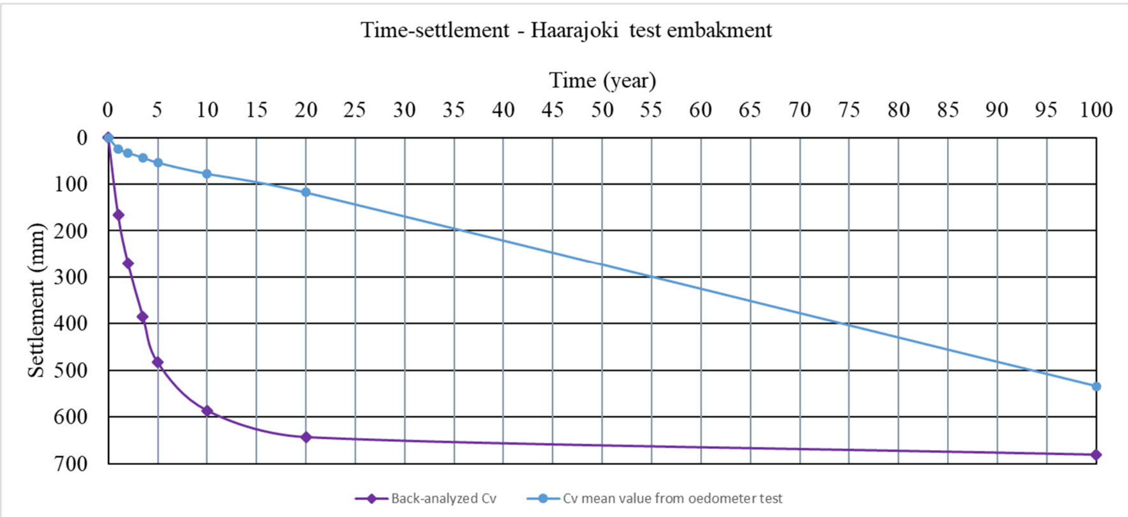


Figure 24. Time-settlement analysis for 100 years using values of C_v of Table 7. Settlements estimated at the centerline of the Haarajoki test embankment.

The considerable conservatism of the values of C_v obtained from oedometer tests can be partly explained by the observations made by Tavenas et al. (1986). According to the authors, oedometer test results underestimate the horizontal and vertical field values of hydraulic conductivity (a property proportional to the soil permeability). Hence, the vertical $(k_v)_f$ and horizontal hydraulic conductivity in field $(k_h)_f$ are equal to the value determined in laboratory tests $(k_v)_l$ and $(k_h)_l$ respectively, multiplied by the ratio of the field value to the laboratory value (C_f). This relation is expressed as follows:

$(k_h)_f = C_f(k_h)_l$ or $(k_v)_f = C_f(k_v)_l$

Tavenas et al. (1986) observed that C_f can be larger than the unity in stratified deposits even with thin intermediate sand lenses, which cannot be detected during sampling. Chai and Miura (1999) recommended using back-analyses to determine $(k_h)_f$ from observed values of settlements of embankments.

The unrealistic time-settlement analysis obtained in Figure 24 can be also explained from the stress dependence of the coefficient of consolidation. Values of C_v do not always remain constant during virgin consolidation (the normal consolidation range) due to the relationship between stress and C_v. In fact, C_v may increase or decrease (Leonards & Ramiah, 1959 in Elkateb, 2017; Ravaska & Vepsäläinen, 2001) or remain as constant (Ravaska & Vepsäläinen 2001) with increasing stress. Likewise, the relationship between C_v and stress depends upon the soil type (Ravaska & Vepsäläinen 2001). For instance, Janbu (1979) observed that for sensitive clays, the value of C_v decreases significantly after the preconsolidation pressure is exceeded but after that it remains nearly constant of increases slowly with the increase in effective stress. Other studies (e.g. Duncan 1993, Terzaghi et al. 1996) observed that C_v is likely to decrease with increasing stress over the normal consolidation range.

In the case of post-glacial soft clays, there is an increase in C_v with increasing stress levels. Moreover, Ravaska and Vepsäläinen (2001) demonstrated that the coefficient of consolidation C_v is constant if the stress-strain relationship is linear and the permeability also remains constant. This is, however, not the case of very soft post-glacial clays whose stress-strain relationship is non-linear. The non-linearity of the stress-strain behavior of clays such as the Haarajoki case has a significant impact not only on the magnitude of settlement but also in the consolidation time. The non-linearity of the strain-permeability has also influence in this C_v behavior. Ravaska and Vepsäläinen (2001) developed a method by which, the permeability at various strain levels is measured after each loading step during an oedometer test. This method allows calculating different C_v values with their corresponding stress states. In the Haarajoki case, when the values of C_v were selected from oedometer tests, these values were assumed constant for all the changes in stress over the virgin compression stress range, neglecting the change in C_v with stress level. Therefore, it is evident that the very conservative values of C_v are not suitable for the applied stress range.

Figure 25 shows the results from time-settlement analysis using the best estimates of values of C_v shown in Table 7 in comparison with the results from characteristic values considering different values of V_x. Abnormal behavior is observed for the first 2.5 years in Figure 25, which is probably caused by some feature in GeoCalc's algorithm.

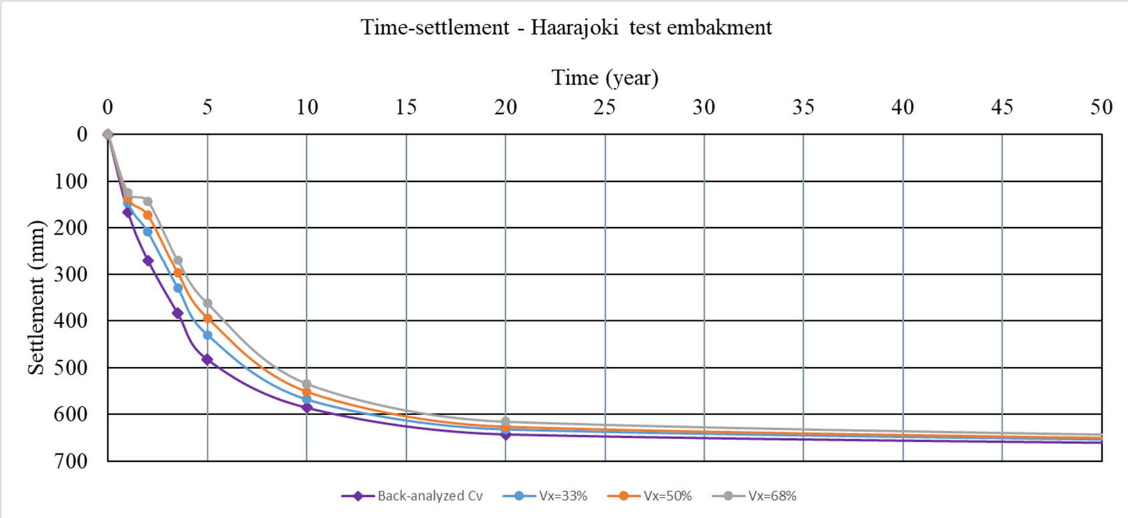


Figure 25. Time-settlement analysis for a 100 years period using best estimates and characteristic values of C_v. Settlements estimated at the centerline of the Haarajoki test embankment.

Table 8 presents the ratios of the consolidation degree obtained through characteristic values of C_v to the results from best estimates. Results are separated according to the coefficient of variation used in the estimation of the characteristic values of C_v, whose values are in the range of 33 to 68%.

Table 8. Ratio of consolidation degree from modified parameters (characteristic values) to values from unmodified parameters (best estimates).

Year	Degree of consolidation [%]				Ratio to best estimate		
	Best estimate [%]	V_x=0.33	V_x =0.50	V_x =0.68	V_x =0.33	V_x =0.50	V_x =0.68
2	39.37	30.52	25.23	20.86	1.3	1.6	1.9
3.5	56.13	48.12	43.37	39.48	1.2	1.3	1.4

Characteristic values of C_v were reduced for a higher coefficient of variation. Therefore, a lower degree of consolidation was obtained for at 2 and 3.5 years with respect to the best estimate for the same periods. However, the selection of the most conservative scenario depends on the design situation, and thus, characteristic values can also yield a higher degree of consolidation with respect to the best estimate.

7 Conclusions

In this thesis, several total settlements analyses were carried out to quantify the margin provided by representative values (i.e., characteristic values) in comparison with values of settlements calculated from the mean of derived values of soil properties (i.e., best estimates). The analyses were made as functions of the range of coefficient of variation reported in the literature for the relevant soil parameters used in the calculations. Likewise, they were separated by calculation method (i.e., deformation model) and by the statistical equation used in the estimation of characteristic values. The unusual number of tests per layer in most of the study cases offered an opportunity to apply the statistically-based equation proposed in October 2019 draft.

These analyses aimed to assess the level of conservatism provided by the characteristic value approach in the calculation of settlements of embankments founded on fine-grained soils. Naturally, with a higher coefficient of variation the values of settlement become more conservative. Values of the coefficient of variation of soil properties reported in the literature are presented as a range of values within which, the coefficient of variation of a specific soil property is likely to fall. The coefficient of variation found in the literature can be used with relative confidence, as it is independent of the geological history of a site. However, these ranges can be very large and yield very different characteristic values in terms of conservatism levels, especially when using the tangent modulus method. Therefore, the definition of a “recommended” value of the coefficient of variation for compressibility parameters is advisable. This recommended value of the coefficient of variation has to be relevant for site conditions in Finland. The significance of finding a recommended coefficient of variation for relevant soil properties is greater considering the conservatism induced by the October 2019 draft equation in cases where limited test results are available. Furthermore, the October 2019 draft recommends selecting a conservative upper estimate of V_X, but the ratios of modified to unmodified obtained for these cases are quite large. Therefore, this recommendation is impractical, especially for the tangent modulus method. In cases where few data is available, usage of Schneider’s equation is highly advisable to avoid large conservatism induced by sampling uncertainty.

The accountability of the tangent modulus method for the nonlinearities of the stress-strain behavior of soft Finnish clays has a great significance in the margin of safety provided by the characteristic values. From the ratios of settlements from modified to settlements from unmodified it is evident that determination of characteristic values fails to provide a uniform margin of safety for all the cases when the tangent modulus method is used. Whilst the safety margin is almost consistent for the majority of the cases, considerably high conservatism was obtained in the Haarajoki case, which is characterized by clays of high compressibility and strong non-linear stress-strain behavior. This is due to the fact that different soil conditions are altering the uncertainty of the same soil parameters that were accounted for among the cases. Moreover, when the modulus number is not adjusted as described in the methodology, the structuration effects are amplified and the margin of safety results even larger. Therefore, there is a need for more guidance to apply the statistically-based equation for characteristic values of soil parameters presented in the October 2019 draft, considering special features of the stress-strain behavior. A direct application of this equation for soil parameters such as m and β will result in too conservative values of settlement, especially in highly structured clays. Additionally, further guidance is needed from EC7 on how to select the soil parameters to which, the procedure to determine the characteristic values is applied. This is of great significance for correlated soil properties, since applying a safety margin to all the parameters involved in the calculation model might result in unnecessary conservatism. Thus, it is also necessary to define a correct interpretation of the determination of characteristic values of soil properties for serviceability limit state verification.

The total factor of safety used in stability analyses was used as a basis to evaluate the conservatism obtained via the application of characteristic values. Ratios of settlements from modified parameters to unmodified parameters were compared to the total factors of safety for embankments. This comparison is only a guide to evaluate the certainty since the consequences of exceeding a serviceability limit state are not as severe as they would be for an ultimate limit state. Based on customary total safety factors reported in the literature for stability analyses of embankments, ratios over 1.5 seem to be already too conservative, especially considering the differences in the consequences of exceeding service and ultimate limit states. The ratios obtained for time-settlement analyses are on the same scale as ratios from the total settlement analyses. Therefore, the same conclusion can be drawn for time-settlement analyses.

Partial factors of safety used in limit states verification are used to provide a safety margin against the occurrence of unfavorable values of soil properties that could not be compensated with the safety margin of the characteristic value. In serviceability limit state partial factors equal to the unity (1.0) are used. A calibration of the partial factor of safety using probabilistic analyses can be performed for further assessment of the certainty margins obtained in this study.

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Appendix

Appendix 1. Tables B.1, B.4 – B.7 from informative Annex B, October 2019 draft.
Appendix 2. Layering of the subsoil for Kujala, Murro and Östersundom cases.
Appendix 3. Values of soil parameters from oedometer test and other tests.
Appendix 4. Calculation example of characteristic values for Tangent Modulus method.
Appendix 5. Best estimates and characteristic values of soil parameters. Tangent Modulus method.
Appendix 6. Best estimates and characteristic values of soil parameters. Compression index method.

Appendix 1. Tables B.1, B.4 – B.7 from informative Annex B, October 2019 draft.

(10) <RCM> The value of k_n should be obtained from Table B.1 which collates Formulas (B.4) to (B.7) for the Cases defined above.

Table B.1 —Values of k_n for different cases and type of estimations

CASES	Case 1: "V_X known & Case 2: "V_X assumed"	Case 3 "V_X unknown"
Case A: Estimate of the mean value	$k_n = N_{95}\sqrt{\frac{1}{n}}$ (B.4)	$k_n = t_{95,n-1}\sqrt{\frac{1}{n}}$ (B.5)
Case B: Estimate of the inferior or superior value (5 or 95 % fractile)	$k_n = N_{95}\sqrt{1+\frac{1}{n}}$ (B.6)	$k_n = t_{95,n-1}\sqrt{1+\frac{1}{n}}$ (B.7)

NOTE 141. Tables B.4 to B.7 collates the values of Normal and Student's t factor (N_{95} or $t_{95,n-1}$) and resulting k_n for the different Cases and type of estimation, according to Formulas (B.4) to (B.7)

Table B.4 — Selected values of N_{95}, $t_{95,n-1}$ and k_n to estimate the characteristic value as the mean value [Case A1 & A2], according to Formula (B.4)

N	2	3	4	5	6	7	8	9	10	12
N_{95}	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
k_n	1.16	0.95	0.82	0.74	0.67	0.62	0.58	0.55	0.52	0.47
N	14	16	18	20	25	30	35	40	50	100
N_{95}	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
k_n	0.44	0.41	0.39	0.37	0.33	0.30	0.28	0.26	0.23	0.16

Table B.5 — Selected values of N_{95}, $t_{95,n-1}$ and k_n to estimate the characteristic value as the mean value [Case A3], according to Formula (B.5)

N	2	3	4	5	6	7	8	9	10	12
$t_{95,n-1}$	6.31	2.92	2.35	2.13	2.02	1.94	1.89	1.86	1.83	1.80
k_n	4.46	1.69	1.18	0.95	0.82	0.73	0.67	0.62	0.58	0.52
n	14	16	18	20	25	30	35	40	50	100
$t_{95,n-1}$	1.77	1.75	1.74	1.73	1.71	1.70	1.69	1.68	1.68	1.66
k_n	0.47	0.44	0.41	0.39	0.34	0.31	0.29	0.27	0.24	0.17

Table B.6 — Selected values of N_{95}, $t_{95,n-1}$ and k_n to estimate the characteristic value as the inferior or superior value (5 or 95% fractile) [Case B1 & B2], according to Formula (B.6)

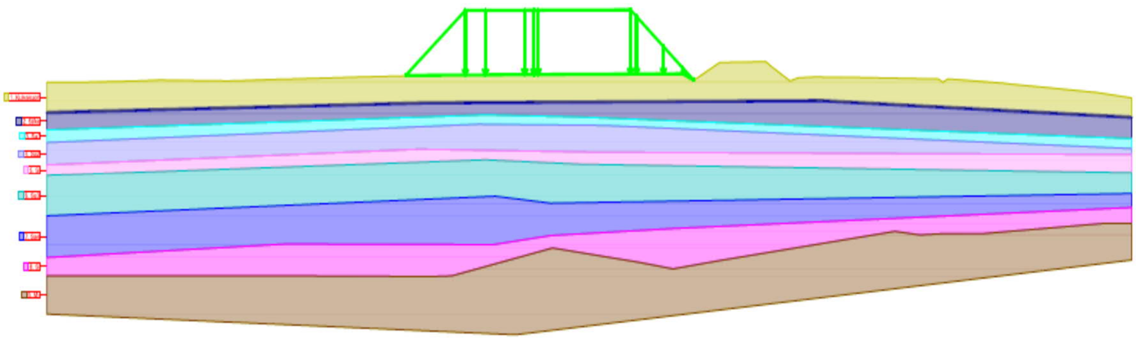
n	2	3	4	5	6	7	8	9	10	12
N_{95}	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
k_n	2.01	1.90	1.84	1.80	1.78	1.76	1.74	1.73	1.73	1.71
n	14	16	18	20	25	30	35	40	50	100
N_{95}	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
k_n	1.70	1.70	1.69	1.69	1.68	1.67	1.67	1.67	1.66	1.65

Table B.7 — Selected values of N_{95}, $t_{95,n-1}$ and k_n to estimate the characteristic value as the inferior or superior value (5 or 95% fractile) [Case B3], according to Formula (B.7)

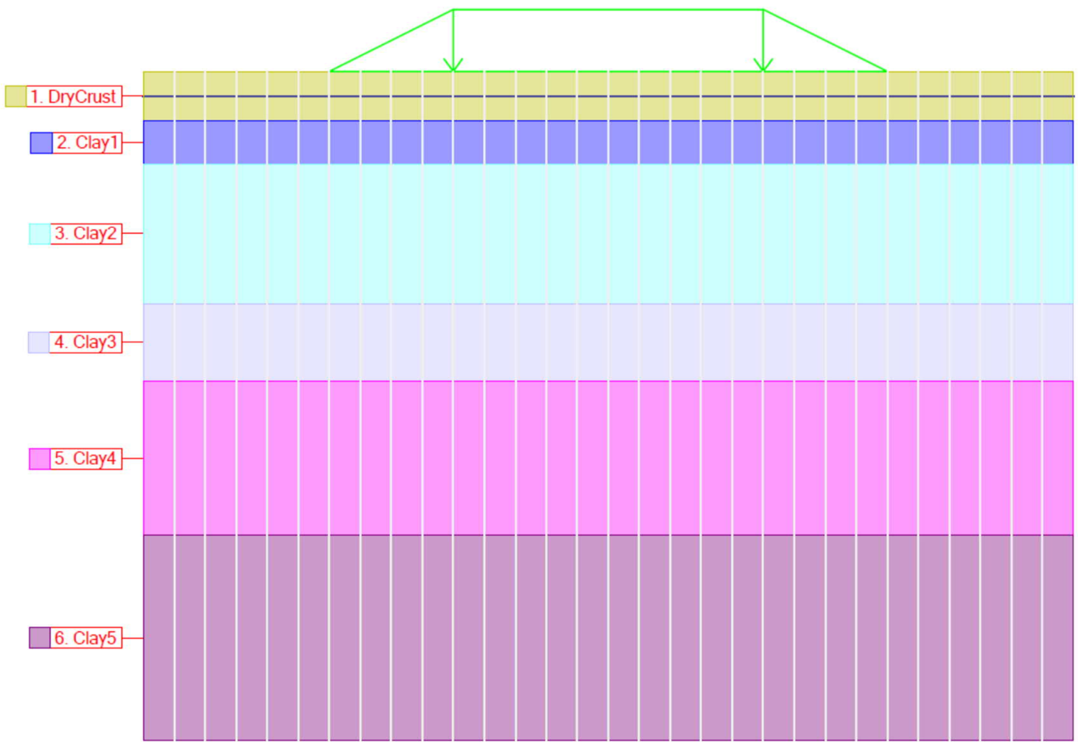
n	2	3	4	5	6	7	8	9	10	12
$t_{95,n-1}$	6.31	2.92	2.35	2.13	2.02	1.94	1.89	1.86	1.83	1.80
k_n	7.73	3.37	2.63	2.34	2.18	2.08	2.01	1.96	1.92	1.87
n	14	16	18	20	25	30	35	40	50	100
$t_{95,n-1}$	1.77	1.75	1.74	1.73	1.71	1.70	1.69	1.68	1.68	1.68
k_n	1.83	1.81	1.79	1.77	1.74	1.73	1.71	1.71	1.69	1.67

Appendix 2. Layering of the subsoil for Kujala, Murro and Östersundom cases

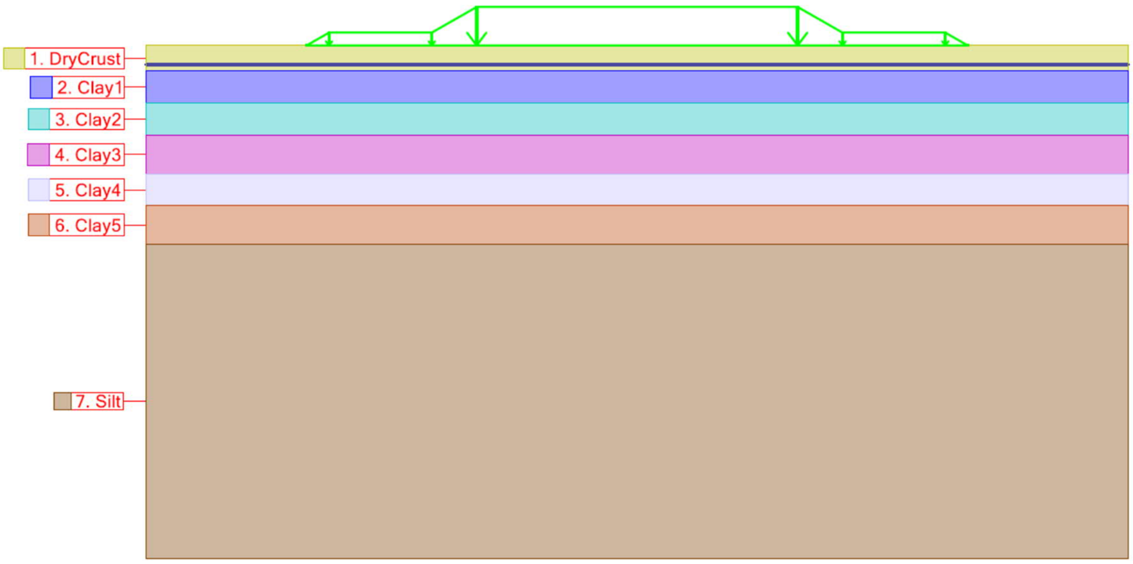
Kujala case (GeoCalc model)



Murro case (GeoCalc model)



Östersundom case (GeoCalc model)



Appendix 3. Values of soil parameters from oedometer test and other tests.

Haraajoki case.

Layer	Depth [m]	Test. No.	Depth sample [m]	γ_0 [kN/m³]	σ'_{v0} [kPa]	σ_p [kPa]	POP [kPa]	m_1 [-]	β_1 [-]	m_2 [-]	β_2 [-]	Note[a]	C_v OC [m²/year]	C_v NC [m²/year]
Dry Crust	0 - 2	1621	0,64 - 0,67	17,68	4	80	-	27,67	0,26	106,53	0,69	CPP	10,23	1,67
			1,25 - 1,28	16,89	9	39	-	28,27	0,354	78	0,878	CPP		
			1,28 - 1,31	17,08	9	44	-	32,73	0,3	301	1,625	CPP		
		1633	1,72 - 1,75	17,35	12	60	-	25,91	0,46	56,66	1,015	CPP	9,4	0,94
		1607	1,77 - 1,8	16,86	12	24	-	27,26	0,65	63,91	0,875	CPP	8,95	0,895
Clay 1	2 - 6	1632	2,31 - 2,34	13,88	15	52	-	4,13	-1,145	60,91	1,24	CPP	0,7	0,07
		1624	3,09 - 3,12	13,83	19	58	-	3,65	-1,24	46,6	0,91	CPP	0,5	0,05
		1604	3,22 - 3,25	13,91	20	49	-	4,32	-0,805			CPP	0,6	0,06
			3,68 - 3,71	13,94	22	51	-	4,51	-0,845	52,1	0,841	CPP		
			3,94 - 3,97	14,42	24	54	-	4,64	-0,612			CPP		
			3,97 - 4	14,09	24	59	-	3,95	-0,875			CPP		
			4,00 - 4,03	13,88	24	49	-	5,08	-0,48			CPP		
		1601	4,34 - 4,37	14,17	26	53	-	4,43	-1,12	50,37	0,675	CPP	0,45	0,045
Clay 2	6 -13	1627V	6,12 - 6,15	14,37	33	90	57	3	-0,895	51,93	1,09	POP	0,85	0,085
			6,34 - 6,37	13,74	34	52	18	4,94	-1,02	59,1	0,775	POP	0,7	0,07
			7,97 - 8	14,84	39	52	13	5,42	-0,43			POP		
			8 - 8,03	14,85	39	68	29	4,4	-0,575			POP		
		1603	8,34 - 8,37	14,99	40	70	30	4,49	-0,73	55,9	0,83	POP	0,5	0,05
		1629	9,03 - 9,06	14,94	43	82	39	5,62	-0,14			POP		
		1631	9,32 - 9,35	15,10	44	100	56	3,18	-0,795	60,01	0,975	POP	0,9	0,09
		1606	11,22 - 11,25	14,48	50	95	45	2,04	-1,255	79,83	0,895	POP	1,4	0,14
		1605	13,22 - 13,25	14,92	57	99	42	2,69	-0,86	72,2	0,87	POP	3,25	0,325

[a]CPP=Constant preconsolidation pressure, POP=Pre-overburden pressure

Layer	Depth [m]	Test. No.	Depth sample [m]	e_0 [-]	C_C [-]	C_S [-]
Dry Crust	0 - 2	1621	0,64 - 0,67	1,06	0,23	0,054
			1,25 - 1,28	1,358	0,18	0,021
			1,28 - 1,31	1,332	0,16	0,021
		1633	1,72 - 1,75	1,338	0,24	0,083
		1607	1,77 - 1,8	1,494	0,53	0,11
Clay 1	2 - 6	1632	2,31 - 2,34	3,287	3,41	0,167
		1624	3,09 - 3,12	3,413	4,05	0,227
		1604	3,22 - 3,25	3,349	3,13	0,195
			3,68 - 3,71	3,165	2,69	0,194
			3,94 - 3,97	2,84		
			3,97 - 4	3,054	2,8	
			4,00 - 4,03	3,074	2,18	
		1601	4,34 - 4,37	3,034	2,99	0,164
Clay 2	6 -13	1627V	6,12 - 6,15	2,535	2,43	0,182
			6,34 - 6,37	3,072	2,52	0,173
			7,97 - 8	2,512	1,64	
			8 - 8,03	2,565	1,58	
		1603	8,34 - 8,37	2,392	1,48	0,186
		1629	9,03 - 9,06	2,322	1,24	0,251
		1631	9,32 - 9,35	2,253	1,67	0,149
		1606	11,22 - 11,25	2,688	2,83	0,153
		1605	13,22 - 13,25	2,537	2,31	0,153

Appendix 3. Values of soil parameters from oedometer test and other tests.

Kujala case

Layer	Depth [m]	Test. No.	Depth sample [m]	γ_0 [kN/m³]	σ'_{v0} [kPa]	σ_p [kPa]	POP [kPa]	m_1 [-]	β_1 [-]	m_2 [-]	β_2 [-]	Note[a]
Dry Crust	0-2	-	-	19,20	-	-		-	-	-	-	Non-C.
Silt 1	2 - 3,2	-	2,10 - 3,20	18,00	32	520	400	1,00	1,00	80,00	0,50	POP
Clay 1	3,2 - 3,9	6589	3,52 - 3,54	16,00	40	329,13	80	8,75	-0,07	42,57	-0,26	POP
Clay 2	3,9 - 6,0	6590	5,50 - 5,52	18,00	55	130,12	80	42,93	0,32	91,26	-0,15	POP
Silt 2	6,0 - 6,9	-	- - -	18,50	63	130,12	NC	100,00	0,30	-	-	NC
Clay 3	6,9 - 10	6591	7,50 - 7,52	18,00	70	43,57	100	49,65	0,38	121,61	-0,08	POP
Clay 4	10 - 12,6	6592	10,50 - 10,52	17,00	90	82,12	100	13,90	0,21	61,52	-0,25	POP
Silt 3	12,6 - 13,5	6593	12,50 - 12,52	16,59	105	79,44	-	20,70	0,40	70,93	-0,27	POP
Morraine	13.5 - 20.1	-	- - -	22,00	-	-	-	-	-	-	-	Non-C.

[a]Non-C=Non compressible

Layer	Depth [m]	Test. No.	Depth sample [m]	e_0 [-]	C_C [-]	C_S [-]
Dry Crust	0-2	-	-	-	-	-
Silt 1	2 - 3,2	-	2,10 - 3,20	-	-	-
Clay 1	3,2 - 3,9	6589	3,52 - 3,54	1,32	0,53	0,10
Clay 2	3,9 - 6,0	6590	5,50 - 5,52	1,04	0,20	0,04
Silt 2	6,0 - 6,9	-	- - -	-	-	-
Clay 3	6,9 - 10	6591	7,50 - 7,52	0,90	0,13	0,03
Clay 4	10 - 12,6	6592	10,50 - 10,52	1,82	0,61	0,09
Silt 3	12,6 - 13,5	6593	12,50 - 12,52	1,63	0,49	0,08
Morraine	13.5 - 20.1	-	- - -	-		

Appendix 3. Values of soil parameters from oedometer test and other tests.

Murro case

Layer	Depth [m]	Test. No.	Depth sample [m]	γ_0 [kN/m^3]	σ'_{v0} [kPa]	σ_p [kPa]	POP [kPa]	m_1 [-]	β_1 [-]	m_2 [-]	β_2 [-]	Note
Dry Crust	0-1,6	1154	0,28 - 0,30	15,85	12,76	90,00	77,24	11,20	0,00	115,73	0,72	POP
		1153	0,30 - 0,30	15,68	12,81	100,00	87,19	10,81	0,05	115,73	0,72	POP
		1166	1,17 - 1,20	16,29	17,00	42,00	25,00	15,98	0,00	164,97	0,52	POP
		1141	1,37 - 1,38	16,17	18,44	100,00	81,56	14,51	0,07	164,97	0,52	POP
		1124V	1,40 - 1,46	16,05	18,73	60,00	41,27	18,50	0,18	164,97	0,52	POP
		1096	1,46 - 1,49	16,14	18,73	133,00	114,27	8,70	-0,09	115,73	0,72	POP
		1123	1,46 - 1,49	16,18	20,00	70,00	50,00	19,91	0,33	115,73	0,72	POP
		1081	1,50 - 1,53	15,96	20,00	100,00	80,00	10,43	0,00	115,73	0,72	POP
Clay 1	1,6-3,0	1162	2,01 - 2,03	15,54	23,00	30,00	NC	13,05	-0,05	NC	NC	NC
		1082	2,60 - 2,63	14,38	26,00	28,00	NC	9,19	0,05	NC	NC	NC
Clay 2	3,0-7,5	1142	3,23 - 3,25	14,42	28,20	42,00	NC	7,13	-0,37	NC	NC	NC
		1097	3,31 - 3,33	14,19	30,00	31,00	NC	8,50	-0,42	NC	NC	NC
		1083	3,57 - 3,60	14,57	31,00	30,00	NC	8,55	-0,19	NC	NC	NC
		1125	3,66 - 3,69	14,21	32,00	28,00	NC	9,35	-0,18	NC	NC	NC
		1084	4,52 - 4,55	14,31	35,00	38,00	NC	6,86	-0,33	NC	NC	NC
		1085	5,57 - 5,60	14,68	40,00	38,00	NC	7,48	-0,23	NC	NC	NC
		1143	6,48 - 6,50	14,76	45,00	40,00	NC	8,63	-0,24	NC	NC	NC
		1127	6,56 - 6,59	14,39	45,00	31,00	NC	8,36	-0,22	NC	NC	NC
		1086	6,62 - 6,65	14,57	45,00	38,00	NC	7,76	-0,20	NC	NC	NC
Clay 3	7,5-10,0	1103	7,58 - 7,60	15,02	49,00	49,00	NC	7,40	-0,52	NC	NC	NC
		1101	9,60 - 9,63	15,83	60,00	55,00	NC	7,42	-0,30	NC	NC	NC
Clay 4	10,0-15,0	1098	10,35 - 10,37	14,95	64,00	64,00	NC	7,50	-0,51	NC	NC	NC
		1087	10,52 - 10,55	15,75	65,00	42,00	NC	9,48	-0,14	NC	NC	NC
		1104	11,58 - 11,60	16,01	71,00	80,00	NC	5,81	-0,62	NC	NC	NC
		1144	12,34 - 12,36	15,73	75,00	83,00	NC	6,15	-0,43	NC	NC	NC
		1129	12,42 - 12,45	15,25	75,00	83,00	NC	6,58	-0,36	NC	NC	NC
		1105	12,48 - 12,50	15,25	76,00	90,00	NC	5,00	-0,44	NC	NC	NC
		1088	13,52 - 13,55	14,05	81,00	50,00	NC	9,39	-0,13	NC	NC	NC
Clay 5		1090	16,60 - 16,63	100,00	68,00	68,00	NC	13,20	-0,08	NC	NC	NC
		1099	17,52 - 17,54	105,00	105,00	105,00	NC	12,50	-0,40	NC	NC	NC
		1089	19,63 - 19,66	118,00	80,00	80,00	NC	8,60	-0,20	NC	NC	NC
		1100	21,58 - 21,61	130,00	130,00	130,00	NC	12,90	-0,27	NC	NC	NC

Appendix 3. Values of soil parameters from oedometer test and other tests.

Murro case

Layer	Depth [m]	Test. No.	Depth sample [m]			e_0 [-]	Cc [-]	Cs [-]
Dry Crust	0-1,6	1154	0,28	-	0,30	1,58	0,541	0,008
		1153	0,30	-	0,30	1,58	0,571	0,007
		1166	1,17	-	1,20	1,45	0,347	0,012
		1193	1,30	-	1,32	1,33	0,365	0,007
		1141	1,37	-	1,38	1,33	0,400	0,009
		1124V	1,40	-	1,46	1,45	0,456	0,007
		1096	1,46	-	1,49	1,59	0,585	0,008
		1123	1,46	-	1,49	1,50	0,503	0,007
		1081	1,50	-	1,53	1,61	0,571	0,019
Clay 1	1,6-3,0	1162	2,01	-	2,03	1,65	0,491	0,018
		1082	2,60	-	2,63	1,82	0,681	0,024
Clay 2	3,0-7,5	1142	3,23	-	3,25	2,49	1,233	0,028
		1097	3,31	-	3,33	2,44	1,111	0,009
		1083	3,57	-	3,60	2,44	0,964	0,036
		1126V	3,60	-	3,66	2,40	1,122	0,009
		1125	3,66	-	3,69	2,29	0,861	0,023
		1084	4,52	-	4,55	2,63	1,388	0,033
		1085	5,57	-	5,60	2,46	1,141	0,044
		1143	6,48	-	6,50	2,14	0,776	0,023
		1128V	6,50	-	6,56	2,27	1,050	0,009
		1127	6,56	-	6,59	2,42	1,014	0,028
		1086	6,62	-	6,65	2,39	1,059	0,038
Clay 3	7,5-10,0	1103	7,58	-	7,60	2,29	1,012	0,009
		1101	9,60	-	9,63	2,04	0,861	0,020
Clay 4	10,0-15,0	1098	10,35	-	10,37	1,69	0,687	0,008
		1087	10,52	-	10,55	1,78	0,651	0,031
		1104	11,58	-	11,60	1,71	0,848	0,020
		1144	12,34	-	12,36	1,70	0,890	0,016
		1130V	12,36	-	12,42	1,73	0,689	0,008
		1129	12,42	-	12,45	1,65	0,660	0,008
		1105	12,48	-	12,50	1,93	1,095	0,030
		1088	13,52	-	13,55	1,86	0,617	0,031
		1102	14,6		14,62	2,266	0,867	0,008
Clay 5	15-21,5	1090	16,6		16,63	1,67	0,598	0,008
		1099	17,515		17,54	1,437	0,496	0,007
		1089	19,63		19,66	1,659	0,657	0,029
		1145	21,43		21,46	1,672	0,657	0,008
		1132V	21,49		21,55	1,551	0,558	0,008
		1131	21,55		21,58	1,34	0,456	0,007

Appendix 3. Values of soil parameters from oedometer test and other tests.

Östersundom case

Layer	Depth [m]	Test. No.	Depth sample [m]	γ_0 [kN/m³]	σ'_{v0} [kPa]	σ_p [kPa]	POP [kPa]	m_1 [-]	β_1 [-]	m_2 [-]	β_2 [-]	Note[a]
Dry crust*	0-0,8	-		17,2	10,32	140	-	100	1	100	1	CPP
Clay 1	0,8-1,8	6255	0,83 - 0,86	16,4	11,76	140	128,2	11,22	0,07	90,9	0,7	POP
		6290	1,03 - 1,06	15,8	11,888	139,7	127,812	7,44	-0,12	46,67	0,36	POP
		6358	1,07 - 1,09	17,09	13,048	127,9	114,852	13,88	-0,13	111,7	0,61	POP
		6222	1,07 - 1,1	15,12	13,29615	79,5	66,20385	7,39	0,11	78,42	-0,37	POP
		6388	1,11 - 1,14	15,85	13,32175	104,9	91,57825	10,83	0,02	148,04	0,85	POP
		6380	1,14 - 1,17	15,67	13,55575	55,1	41,54425	10,51	0,06	94,57	0,89	POP
		6249	1,19 - 1,22	15	13,72585	35,1	21,37415	6,08	-0,4	52,59	0,65	POP
		6238	1,27 - 1,3	16,8	13,97585	57	43,02415	9,29	-0,21	65,76	0,45	POP
		6291	1,37 - 1,4	14,9	14,51985	78,9	64,38015	7,99	-0,19	46,96	0,55	POP
		6258	1,37 - 1,4	16,2	15,00985	30,8	15,79015	9,83	-0,28	83,65	0,41	POP
		6376	1,55 - 1,57	14,86	15,00985	65,8	50,79015	7,43	-0,28	47,08	0,33	POP
		6243	1,77 - 1,8	14,53	15,86035	31,2	15,33965	7,75	-0,33	45,68	0,37	POP
Clay 2	1,8-2,8	6237	1,98 - 2,01	14,43	16,8796	30,5	13,6204	5,69	-0,86	52,11	0,41	POP
		6254	2,23 - 2,26	15,1	17,8099	45,5	27,6901	8,3	-0,19	53,81	0,38	POP
		6233	2,29 - 2,32	13,47	19,0849	39,2	20,1	9,55	-1,09	56,94	-0,35	POP
		6387	2,31 - 2,34	16,05	19,2931	21,7	2,4	10,8	-0,22	127,59	0,7	POP
		6223	2,47 - 2,5	14,94	19,4141	21,2	1,7859	7,58	-0,54	86,9	-0,28	POP
		6225	2,77 - 2,8	14,71	20,2045	30,8	10,5955	7,3	-0,44	37,72	0,32	POP
Clay 3	2,8-4,0	6374	2,86 - 2,89	16,17	21,6175	33,5	0	10,53	-0,15	85,01	0,42	NC
		6257	2,87 - 2,9	14,8	22,1728	19,3	0	8,07	-0,47	45,55	0,41	NC
		6234	2,94 - 2,97	13,7	22,2208	25,1	2,8792	7,42	-1,03	42	0,27	NC
		6390	3 - 3,2	15,3	22,4798	26,4	3,9202	7,53	-0,15	71,2	-0,23	NC
		6235	3,64 - 3,67	15	23,2483	31,4	8,1517	9,1	-0,68	54,68	-0,28	NC
		6356	3,88 - 3,9	14,91	26,0233	23,6	0	8,96	-0,24	61,26	0,43	NC
Clay 4	4,0-5,0	6248	4,07 - 4,1	16,6	27,17715	23,6						POP
		6250	4,2 - 4,23	16	28,46415		0	9,23	-0,29	100,38	-0,36	POP
		6256	4,2 - 4,23	14,2	29,24415	31,4	2,15585	10,3	-0,04	51,71	0,33	POP
		6375	4,34 - 4,36	15,82	29,24415	23	0	8,97	-0,28	56,83	0,38	POP
		6357	4,52 - 4,53	19,61	30,02985	23,6	0	64,74	0,23	217,64	0,06	POP
		6389	4,72 - 4,75	17,6	31,71	58,6	26,8884	10,62	-0,19	167,65	0,67	POP
		6226	4,77 - 4,80	17,79	33,3076	36,10	2,7924	12,75	-0,51	126,73	0,51	POP
Clay 5	5,0-6,12	6251	5,06 - 5,09	18,57	33,6971	56,24	22,5429					POP
		6224	5,07 - 5,1	17,7	36,1824		-	9,77	-0,42	149,94	-0,81	POP
		6381	5,44 - 5,47	17,98	36,2594	45,52	9,2606	12,87	-0,15	183,49	0,83	POP
		6304	5,72 - 5,75	18,3	39,212	38,2	0	17,1	-0,02	162,96	-0,3	POP
		6242	5,76 - 5,79	19	41,536	72,3	30,764	16,66	-0,13	166,99	0,64	POP
		6378	6,11 - 6,13	18,7	41,896	67,8	25,904	16,89	-0,05	185,05	0,6	POP
Silt/Sand*	15-21,5	-	- - -	18,7	44,8975	81,9	NC	100	0,3	NC	NC	NC

*No oedometer tests are available for samples of these layers. The values of the parameters of the silt-sand layer were chosen from the ranges presented by Länsivaara (2000).

Appendix 3. Values of soil parameters from oedometer test and other tests.

Östersundom case

Layer	Depth [m]	Test. No.	Depth sample [m]			e_0 [-]	Cc [-]	Cs [-]
Dry crust	0-0,8	-						
Clay 1	0,8-1,8	6255	0,83	-	0,86	1,66	0,59	0,10
		6290	1,03	-	1,06	1,98	0,76	0,14
		6358	1,07	-	1,09	1,38	0,33	0,05
		6222	1,07	-	1,1	2,43	1,16	0,10
		6388	1,11	-	1,14	1,87	0,63	0,10
		6380	1,14	-	1,17	1,97	0,65	0,13
		6249	1,19	-	1,22	2,42	1,51	0,17
		6238	1,27	-	1,3	1,49	0,55	0,08
		6291	1,37	-	1,4	2,52	1,16	0,15
		6258	1,37	-	1,4	1,71	0,62	0,05
		6376	1,55	-	1,57	2,53	1,44	0,17
		6243	1,77	-	1,8	2,78	1,30	0,15
Clay 2	1,8-2,8	6237	1,98	-	2,01	2,85	1,99	0,13
		6254	2,23	-	2,26	2,29	1,00	0,11
		6233	2,29	-	2,32	4,07	4,64	0,02
		6387	2,31	-	2,34	1,83	0,80	0,10
		6223	2,47	-	2,5	2,46	1,74	0,11
		6225	2,77	-	2,8	2,68	1,33	0,17
Clay 3	2,8-4,0	6374	2,86	-	2,89	1,83	0,76	0,09
		6257	2,87	-	2,9	2,74	1,99	0,15
		6234	2,94	-	2,97	3,79	3,91	0,21
		6390	3	-	3,2	2,31	1,04	0,11
		6235	3,64	-	3,67	2,52	1,81	0,17
		6356	3,88	-	3,9	2,53	1,13	0,14
Clay 4	4,0-5,0	6248	4,07	-	4,1	1,55	0,52	0,06
		6250	4,2	-	4,23	1,90	0,93	0,08
		6256	4,2	-	4,23	2,38	0,70	0,12
		6375	4,34	-	4,36	2,00	0,95	0,11
		6357	4,52	-	4,53	0,78	0,07	0,02
		6389	4,72	-	4,75	1,27	0,48	0,04
		6226	4,77	-	4,80	1,16	0,42	0,03
Clay 5	5,0-6,12	6251	5,06	-	5,09	0,92	0,11	0,02
		6224	5,07	-	5,1	1,25	0,92	0,06
		6381	5,44	-	5,47	1,11	0,38	0,05
		6304	5,72	-	5,75	1,03	0,27	0,03
		6242	5,76	-	5,79	0,98	0,24	0,03
		6378	6,11	-	6,13	0,92	0,26	0,04
Silt/Sand*	15-21,5	-	-	-	-			

Appendix 4. Calculation example of characteristic values for Tangent Modulus method according to October 2019 draft.

Layers	Depth	Test Values			Mean value			Vx			(See: Appendix 1, Table B.4)			Characteristic Value - Lower bound of cautious estimate		
		m_1	β_1	σ_p	m_1	β_1	σ_p	m_1	β_1	σ_p	m_1	β_1	σ_p	m_1	β_1	σ_p
2-6 m (Clay 1)	2,31	4,13	-1,15	52	4,34	-0,89	53,13	0,2	0,2	0,10	0,74	0,74	0,74	3,835	-0,993	50,044
	3,09	3,65	-1,24	58				0,4	0,4	0,23				3,332	-1,096	46,038
	3,22	4,32	-0,81	49				0,7	0,7	0,35				2,577	-1,251	42,341
	3,68	4,51	-0,85	51												
	3,94	4,64	-0,61	54												
	3,97	3,95	-0,88	59												
	4	5,08	-0,48	49												
	4,34	4,43	-1,12	53												
Collecting test values of relevant parameters per layer					Calculating mean from the test values			Defining coefficients of variation to be used for each parameter (One Vx value in practice)			Defining Kn based on the coefficient of variation (See: Appendix 1, Table B.4)			Applying Equation (20) for each Vx. Lower values of m, β and σ_p than the mean values yield more conservative results, so a negative symbol (-) in Equation 20 is used for m and σ_p and positive (+) for β		

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Haarajoki case

Settlements - Tangent Modulus Method
Best Estimates of soil parameters

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1b	0,0	0,0	0 - 2	28,37	0,40	121,22	1,02	-	49,40	17,17	685
Clay 1				2 - 6	4,34	-0,89	52,50	0,92	-	53,13	14,02	
Clay 2				6 - 13	3,98	-0,74	63,16	0,91	36,56	-	14,69	

Settlements - Tangent Modulus Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0.20	0.10	0 - 2	24.17	0.34	103.28	0.87	-	45.74	17.17	918
Clay 1				2 - 6	3.84	-0.99	43.89	0.77	-	50.04	14.02	
Clay 2				6 - 13	3.54	-0.83	54.70	0.78	34.55	-	14.69	
Dry Crust	Analysis 2b	0.40	0.23	0 - 2	19.97	0.28	85.34	0.72	-	40.99	17.17	1274
Clay 1				2 - 6	3.33	-1.10	35.28	0.62	-	46.04	14.02	
Clay 2				6 - 13	3.10	-0.91	46.23	0.66	31.93	-	14.69	
Dry Crust	Analysis 2c	0.70	0.35	0 - 2	13.67	0.20	58.43	0.49	-	36.61	17.17	1957
Clay 1				2 - 6	2.58	-1.25	22.36	0.39	-	42.34	14.02	
Clay 2				6 - 13	2.44	-1.03	33.54	0.48	29.52	-	14.69	

Settlements - Tangent Modulus Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0.20	0.10		32.57	0.46	139.16	1.17	-	53.06	17.17	509
Clay 1					4.84	-0.79	61.10	1.07	-	56.21	14.02	
Clay 2					4.41	-0.66	71.63	1.03	38.57	-	14.69	
Dry Crust	Analysis 2b	0.40	0.23		36.76	0.52	157.10	1.32	-	57.81	17.17	355
Clay 1					5.35	-0.68	69.71	1.22	-	60.21	14.02	
Clay 2					4.85	-0.58	80.09	1.15	41.18	-	14.69	
Dry Crust	Analysis 2c	0.70	0.35		43.06	0.61	184.01	1.54	-	62.19	17.17	229
Clay 1					6.10	-0.53	82.63	1.44	-	63.91	14.02	
Clay 2					5.51	-0.46	92.78	1.33	43.59	-	14.69	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Haarajoki case

Settlements - Tangent Modulus Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0.20	0.10	0 - 2	25.53	0.36	109.10	0.91	-	46.93	17.17	
Clay 1				2 - 6	3.90	-0.98	47.25	0.82	-	50.47	14.02	878
Clay 2				6 - 13	3.58	-0.82	56.85	0.82	34.73	-	14.69	
Dry Crust	Analysis 4b	0.40	0.23	0 - 2	22.69	0.32	96.98	0.81	-	43.72	17.17	
Clay 1				2 - 6	3.47	-1.07	42.00	0.73	-	47.02	14.02	1161
Clay 2				6 - 13	3.18	-0.89	50.53	0.72	32.35	-	14.69	
Dry Crust	Analysis 4c	0.70	0.35	0 - 2	18.44	0.26	78.79	0.66	-	40.76	17.17	
Clay 1				2 - 6	2.82	-1.20	34.12	0.60	-	43.83	14.02	1648
Clay 2				6 - 13	2.58	-1.01	41.06	0.59	30.16	-	14.69	

Settlements - Tangent Modulus Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0.20	0.10	0 - 2	31.20	0.45	133.34	1.12	-	51.87	17.17	
Clay 1				2 - 6	4.77	-0.80	57.74	1.01	-	55.78	14.02	533
Clay 2				6 - 13	4.37	-0.67	69.48	1.00	38.38		14.69	
Dry Crust	Analysis 4b	0.40	0.23	0 - 2	34.04	0.49	145.46	1.22	-	55.08	17.17	
Clay 1				2 - 6	5.21	-0.71	62.99	1.10	-	59.23	14.02	395
Clay 2				6 - 13	4.77	-0.60	75.79	1.09	40.76		14.69	
Dry Crust	Analysis 4c	0.70	0.35	0 - 2	38.30	0.55	163.65	1.37	-	58.05	17.17	
Clay 1				2 - 6	5.86	-0.58	70.87	1.24	-	62.42	14.02	275
Clay 2				6 - 13	5.37	-0.48	85.27	1.22	42.95		14.69	

Kujala case

Settlements - Tangent Modulus Method
Best Estimates of soil parameters

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1b	0,0	0,0	0-2	-	-	-	-		-	19,20	
Silt 1				2 - 3,2	1,00	1,00	80,00	0,50	400,00	-	18,09	
Clay 1				3,2 - 3,9	8,75	-0,07	42,57	-0,26	80,00	-	16,08	
Clay 2				3,9 - 6,0	42,93	0,32	91,26	-0,15	80,00	-	18,09	
Silt 2				6,0 - 6,9	100,00	0,30	NC	NC	NC	-	18,59	122
Clay 3				6,9 - 10	49,65	0,38	121,61	-0,08	100,00	-	18,09	
Clay 4				10 - 12,6	13,90	0,21	61,52	-0,25	100,00	-	17,09	
Silt 3				12,6 - 13,5	20,70	0,40	70,93	-0,27	100,00	-	16,67	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Kujala case

Settlements - Tangent Modulus Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	0,90	0,90	72,00	0,45	380,00	17,91	
Clay 1				3,2 - 3,9	7,88	-0,08	38,31	-0,29	76,00	15,920	
Clay 2				3,9 - 6,0	38,63	0,29	82,13	-0,17	76,00	17,910	138
Silt 2				6,0 - 6,9	90,00	0,27	NC	NC	NC	18,408	
Clay 3				6,9 - 10	44,68	0,35	109,45	-0,08	95,00	17,910	
Clay 4				10 - 12,6	12,51	0,19	55,37	-0,27	95,00	16,915	
Silt 3				12,6 - 13,5	18,63	0,36	63,83	-0,30	95,00	16,506	
Dry Crust	Analysis 4b	0,40	0,23	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	0,80	0,80	64,00	0,40	354,00	17,91	
Clay 1				3,2 - 3,9	7,00	-0,08	34,06	-0,32	70,80	15,92	
Clay 2				3,9 - 6,0	34,34	0,26	73,01	-0,18	70,80	17,91	159
Silt 2				6,0 - 6,9	80,00	0,24	NC	NC	NC	18,41	
Clay 3				6,9 - 10	39,72	0,31	97,29	-0,09	88,50	17,91	
Clay 4				10 - 12,6	11,12	0,17	49,22	-0,30	88,50	16,92	
Silt 3				12,6 - 13,5	16,56	0,32	56,74	-0,33	88,50	16,51	
Dry Crust	Analysis 4c	0,70	0,35	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	0,65	0,65	52,00	0,33	330,00	17,91	
Clay 1				3,2 - 3,9	5,69	-0,09	27,67	-0,36	66,00	15,92	
Clay 2				3,9 - 6,0	27,90	0,21	59,32	-0,21	66,00	17,91	202
Silt 2				6,0 - 6,9	65,00	0,20	NC	NC	NC	18,41	
Clay 3				6,9 - 10	32,27	0,25	79,05	-0,10	82,50	17,91	
Clay 4				10 - 12,6	9,04	0,14	39,99	-0,33	82,50	16,92	
Silt 3				12,6 - 13,5	13,46	0,26	46,10	-0,37	82,50	16,51	

Settlements - Tangent Modulus Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	1,10	1,10	88,00	0,55	420,00	17,91	
Clay 1				3,2 - 3,9	9,629	-0,062	46,829	-0,237	84,000	15,920	
Clay 2				3,9 - 6,0	47,218	0,355	100,385	-0,139	84,000	17,910	109
Silt 2				6,0 - 6,9	110,000	0,330	NC	NC	NC	18,408	
Clay 3				6,9 - 10	54,612	0,423	133,774	-0,068	105,000	17,910	
Clay 4				10 - 12,6	15,293	0,229	67,677	-0,222	105,000	16,915	
Silt 3				12,6 - 13,5	22,774	0,445	78,019	-0,246	105,000	16,506	
Dry Crust	Analysis 4b	0,40	0,23	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	1,20	1,20	96,00	0,60	446,00	17,91	
Clay 1				3,2 - 3,9	10,50	-0,08	51,09	-0,32	89,20	15,92	
Clay 2				3,9 - 6,0	51,51	0,39	109,51	-0,18	89,20	17,91	96
Silt 2				6,0 - 6,9	120,00	0,36	NC	NC	NC	18,41	
Clay 3				6,9 - 10	59,58	0,46	145,94	-0,09	111,50	17,91	
Clay 4				10 - 12,6	16,68	0,25	73,83	-0,30	111,50	16,92	
Silt 3				12,6 - 13,5	24,84	0,49	85,11	-0,33	111,50	16,51	
Dry Crust	Analysis 4c	0,70	0,35	0-2	-	-	-	-	-	19,20	
Silt 1				2 - 3,2	1,35	1,35	108,00	0,68	470,00	17,91	
Clay 1				3,2 - 3,9	11,82	-0,09	57,47	-0,36	94,00	15,92	
Clay 2				3,9 - 6,0	57,95	0,44	123,20	-0,21	94,00	17,91	83
Silt 2				6,0 - 6,9	135,00	0,41	NC	NC	NC	18,41	
Clay 3				6,9 - 10	67,02	0,52	164,18	-0,10	117,50	17,91	
Clay 4				10 - 12,6	18,77	0,28	83,06	-0,33	117,50	16,92	
Silt 3				12,6 - 13,5	27,95	0,55	95,75	-0,37	117,50	16,51	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Murro case

Settlements - Tangent Modulus Method
Best Estimates of soil parameters

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 1b	0,0	0,0	0-1,6	13,76	0,07	134,20	0,64	69,57	16,08	
Clay 1				1,6-3,0	11,12	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	8,07	-0,26	NC	NC	NC	14,45	1044
Clay 3				7,5-10	7,41	-0,41	NC	NC	NC	15,43	
Clay 4				10,0-15,0	7,13	-0,37429	NC	NC	NC	15,41	
Clay 5				15-21,5	11,8	-0,235	NC	NC	NC	16,37	

Settlements - Tangent Modulus Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0,20	0,10	0-1,6	12,16	0,06	118,63	0,57	65,53	16,04	
Clay 1				1,6-3,0	8,54	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	7,18	-0,29	NC	NC	NC	14,46	1253
Clay 3				7,5-10	5,69	-0,51	NC	NC	NC	15,43	
Clay 4				10,0-15,0	6,25	-0,42	NC	NC	NC	15,28	
Clay 5				15-21,5	9,86	-0,27	NC	NC	NC	16,28	
Dry Crust	Analysis 2b	0,40	0,23	0-1,6	10,56	0,05	103,06	0,49	60,29	16,04	
Clay 1				1,6-3,0	5,96	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	6,29	-0,32	NC	NC	NC	14,46	1585
Clay 3				7,5-10	3,97	-0,60	NC	NC	NC	15,43	
Clay 4				10,0-15,0	5,36	-0,47	NC	NC	NC	15,28	
Clay 5				15-21,5	7,93	-0,31	NC	NC	NC	16,28	
Dry Crust	Analysis 2c	0,70	0,35	0-1,6	8,17	0,04	79,71	0,38	55,44	16,04	
Clay 1				1,6-3,0	2,09	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	4,96	-0,37	NC	NC	NC	14,46	3081
Clay 3				7,5-10	1,39	-0,74	NC	NC	NC	15,43	
Clay 4				10,0-15,0	4,04	-0,54	NC	NC	NC	15,28	
Clay 5				15-21,5	5,03	-0,37	NC	NC	NC	16,28	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Murro case

Settlements - Tangent Modulus Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis	V_x m, β	V_x σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0,20	0,10	0-1,6	15,35	0,07	149,76	0,72	65,53	16,04	
Clay 1				1,6-3,0	13,70	0,06	NC	NC	NC	14,96	
Clay 2				3,0-7,5	8,96	0,00	NC	NC	NC	14,46	864
Clay 3				7,5-10	9,13	-0,24	NC	NC	NC	15,43	
Clay 4				10,0-15,0	8,01	-0,31	NC	NC	NC	15,28	
Clay 5				15-21,5	13,74	-0,33	NC	NC	NC	16,28	
Dry Crust	Analysis 2b	0,40	0,23	0-1,6	16,95	0,08	165,33	0,79	60,29	16,04	
Clay 1				1,6-3,0	16,28	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	9,84	-0,21	NC	NC	NC	14,46	802
Clay 3				7,5-10	10,85	-0,22	NC	NC	NC	15,43	
Clay 4				10,0-15,0	8,90	-0,28	NC	NC	NC	15,28	
Clay 5				15-21,5	7,93	-0,16	NC	NC	NC	16,28	
Dry Crust	Analysis 2c	0,70	0,35	0-1,6	19,34	0,09	188,68	0,90	55,44	16,04	
Clay 1				1,6-3,0	20,15	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	11,18	-0,16	NC	NC	NC	14,46	664
Clay 3				7,5-10	13,43	-0,08	NC	NC	NC	15,43	
Clay 4				10,0-15,0	10,22	-0,21	NC	NC	NC	15,28	
Clay 5				15-21,5	18,57	-0,10	NC	NC	NC	16,28	

Settlements - Tangent Modulus Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	V_x m, β	V_x σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-1,6	12,38	0,06	120,78	0,58	66,09	16,04	
Clay 1				1,6-3,0	10,01	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	7,26	-0,29	NC	NC	NC	14,46	1170
Clay 3				7,5-10	6,67	-0,45	NC	NC	NC	15,43	
Clay 4				10,0-15,0	6,42	-0,41	NC	NC	NC	15,28	
Clay 5				15-21,5	10,62	-0,26	NC	NC	NC	16,28	
Dry Crust	Analysis 4b	0,40	0,23	0-1,6	11,00	0,05	107,36	0,51	61,57	16,04	
Clay 1				1,6-3,0	8,90	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	6,46	-0,32	NC	NC	NC	14,46	1327
Clay 3				7,5-10	5,93	-0,49	NC	NC	NC	15,43	
Clay 4				10,0-15,0	5,70	-0,45	NC	NC	NC	15,28	
Clay 5				15-21,5	9,44	-0,28	NC	NC	NC	16,28	
Dry Crust	Analysis 4c	0,70	0,35	0-1,6	8,94	0,04	87,23	0,42	57,39	16,04	
Clay 1				1,6-3,0	7,23	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	5,24	-0,36	NC	NC	NC	14,46	1655
Clay 3				7,5-10	4,82	-0,55	NC	NC	NC	15,43	
Clay 4				10,0-15,0	4,63	-0,51	NC	NC	NC	15,28	
Clay 5				15-21,5	7,67	-0,32	NC	NC	NC	16,28	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Murro case

Settlements - Tangent Modulus Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-1,6	15,13	0,07	147,61	0,71	66,09	16,04	941
Clay 1				1,6-3,0	12,23	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	8,88	-0,24	NC	NC	NC	14,46	
Clay 3				7,5-10	8,15	-0,37	NC	NC	NC	15,43	
Clay 4				10,0-15,0	7,84	-0,34	NC	NC	NC	15,28	
Clay 5				15-21,5	12,98	-0,21	NC	NC	NC	16,28	
Dry Crust	Analysis 4b	0,40	0,23	0-1,6	16,51	0,08	161,03	0,77	61,57	16,04	856
Clay 1				1,6-3,0	13,34	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	9,68	-0,21	NC	NC	NC	14,46	
Clay 3				7,5-10	8,89	-0,33	NC	NC	NC	15,43	
Clay 4				10,0-15,0	8,56	-0,30	NC	NC	NC	15,28	
Clay 5				15-21,5	14,16	-0,19	NC	NC	NC	16,28	
Dry Crust	Analysis 4c	0,70	0,35	0-1,6	18,57	0,09	181,16	0,87	57,39	16,04	752
Clay 1				1,6-3,0	15,01	0,00	NC	NC	NC	14,96	
Clay 2				3,0-7,5	10,89	-0,17	NC	NC	NC	14,46	
Clay 3				7,5-10	10,00	-0,27	NC	NC	NC	15,43	
Clay 4				10,0-15,0	9,63	-0,24	NC	NC	NC	15,28	
Clay 5				15-21,5	15,93	-0,15	NC	NC	NC	16,28	

Östersundom case

Settlements - Tangent Modulus Method
Best Estimates of soil parameters

Layer	Analysis	Vx m, β	Vx σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1b	0,0	0,0	0-0,8	100,00	1,00	100,00	1,00	-	140,00	17,20	249
				0,8-1,8	9,14	-0,14	76,00	0,48	55,53	-	15,69	
				1,8-2,8	8,20	-0,56	69,18	0,20	12,41	-	14,78	
				2,8-4,0	8,60	-0,45	NC	NC	NC	-	14,98	
				4,0-5,0	19,44	-0,18	120,16	0,27	9,06	-	16,80	
				5,0-6,12	14,66	-0,15	169,69	0,19	20,59	-	18,38	
				6,12-16,4	100,00	0,30	NC	NC	NC	-	18,38	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Östersundom case

Settlements - Tangent Modulus Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0,20	0,10	0-0,8	90,00	0,90	90,00	1,00	-	133,00*	17,20	
Clay 1				0,8-1,8	8,28	-0,15	68,86	0,44	52,92	-	15,69	
Clay 2				1,8-2,8	7,10	-0,63	59,91	0,17	11,58	-	14,78	
Clay 3				2,8-4,0	7,45	-0,51	NC	NC	NC	-	14,98	303
Clay 4				4,0-5,0	16,83	-0,20	104,06	0,23	8,46	-	16,80	
Clay 5				5,0-6,12	12,49	-0,18	144,57	0,16	19,06	-	18,38	
Silt/Morr.				6,12-16,4	90,00	0,27	NC	NC	NC	-	18,38	
Dry Crust	Analysis 2b	0,40	0,23	0-0,8	80,00	0,80	80,00	1,00	-	123,90	17,20	
Clay 1				0,8-1,8	7,42	-0,17	61,71	0,39	49,52	-	15,69	
Clay 2				1,8-2,8	6,00	-0,71	50,64	0,14	10,50	-	14,78	
Clay 3				2,8-4,0	6,30	-0,57	NC	NC	NC	-	14,98	385
Clay 4				4,0-5,0	14,23	-0,23	87,95	0,19	7,67	-	16,80	
Clay 5				5,0-6,12	10,32	-0,20	119,46	0,14	17,08	-	18,38	
Silt/Morr.				6,12-16,4	80,00	0,24	NC	NC	NC	-	18,38	
Dry Crust	Analysis 2c	0,70	0,35	0-0,8	65,00	0,65	65,00	1,00	-	115,5	17,20	
Clay 1				0,8-1,8	6,13	-0,19	51,00	0,32	46,39	-	15,69	
Clay 2				1,8-2,8	4,36	-0,82	36,73	0,10	9,50	-	14,78	
Clay 3				2,8-4,0	4,57	-0,67	NC	NC	NC	-	14,98	578
Clay 4				4,0-5,0	10,32	-0,26	63,80	0,14	6,94	-	16,80	
Clay 5				5,0-6,12	7,07	-0,23	81,79	0,09	15,25	-	18,38	
Silt/Morr.				6,12-16,4	65,00	0,20	NC	NC	NC	-	18,38	

Settlements - Tangent Modulus Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 2a	0,20	0,10		110,00	1,10	110,00	1,00	-	147,00	17,20	
Clay 1					10,00	-0,13	83,15	0,53	58,14	-	15,69	
Clay 2					9,30	-0,48	78,45	0,22	13,24	-	14,78	
Clay 3					9,75	-0,39	NC	NC	NC	-	14,98	204
Clay 4					22,04	-0,16	136,26	0,30	9,67	-	16,80	
Clay 5					16,83	-0,13	194,80	0,22	22,11	-	18,38	
Silt/Morr.					110,00	0,33	NC	NC	NC	-	18,38	
Dry Crust	Analysis 2b	0,40	0,23		120,00	1,20	120,00	1,00	-	156,10	17,20	
Clay 1					10,85	-0,11	90,29	0,57	61,53	-	15,69	
Clay 2					10,40	-0,41	87,72	0,25	14,33	-	14,78	
Clay 3					10,91	-0,33	NC	NC	NC	-	14,98	176
Clay 4					24,64	-0,13	152,36	0,34	10,46	-	16,80	
Clay 5					19,00	-0,11	219,91	0,25	24,09	-	18,38	
Silt/Morr.					120,00	0,36	NC	NC	NC	-	18,38	
Dry Crust	Analysis 2c	0,70	0,35		135,00	1,35	135,00	1,00		164,5	17,20	
Clay 1					12,14	-0,09	101,01	0,64	64,66	-	15,69	
Clay 2					12,05	-0,30	101,62	0,29	15,32	-	14,78	
Clay 3					12,64	-0,24	NC	NC	NC	-	14,98	136
Clay 4					28,55	-0,10	176,51	0,39	11,19	-	16,80	
Clay 5					22,25	-0,07	257,58	0,29	25,92	-	18,38	
Silt/Morr.					135,00	0,41	NC	NC	NC	-	18,38	

Appendix 5. Best estimates and characteristic values of soil parameters used in Tangent Modulus method

Östersundom case

Settlements - Tangent Modulus Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-0,8	90,00	0,90	90,00	1,00		133,00*	17,20	
Clay 1				0,8-1,8	8,22	-0,15	68,40	0,44	52,75	-	15,69	
Clay 2				1,8-2,8	7,38	-0,61	62,26	0,18	11,79	-	14,78	
Clay 3				2,8-4,0	7,74	-0,50	NC	NC	NC	-	14,98	287
Clay 4				4,0-5,0	17,49	-0,20	108,14	0,24	8,61	-	16,80	
Clay 5				5,0-6,12	13,19	-0,17	152,72	0,17	19,56	-	18,38	
Silt/Morr.				6,12-16,4	90,00	0,27	NC	NC	NC	-	18,38	
Dry Crust	Analysis 4b	0,40	0,23	0-0,8	80,00	0,80	80,00	1,00		123,90	17,20	
Clay 1				0,8-1,8	7,31	-0,17	60,80	0,39	49,14	-	15,69	
Clay 2				1,8-2,8	6,56	-0,67	55,34	0,16	10,99	-	14,78	
Clay 3				2,8-4,0	6,88	-0,54	NC	NC	NC	-	14,98	342
Clay 4				4,0-5,0	15,55	-0,22	96,13	0,21	8,02	-	16,80	
Clay 5				5,0-6,12	11,73	-0,18	135,75	0,15	18,22	-	18,38	
Silt/Morr.				6,12-16,4	80,00	0,24	NC	NC	NC	-	18,38	
Dry Crust	Analysis 4c	0,70	0,35	0-0,8	65,00	0,65	65,00	1,00		115,5	17,20	
Clay 1				0,8-1,8	5,94	-0,19	49,40	0,31	45,81	-	15,69	
Clay 2				1,8-2,8	5,33	-0,75	44,97	0,13	10,24	-	14,78	
Clay 3				2,8-4,0	5,59	-0,61	NC	NC	NC	-	14,98	451
Clay 4				4,0-5,0	12,63	-0,24	78,10	0,17	7,48	-	16,80	
Clay 5				5,0-6,12	9,53	-0,21	110,30	0,12	16,98	-	18,38	
Silt/Morr.				6,12-16,4	65,00	0,20	NC	NC	NC	-	18,38	

Settlements - Tangent Modulus Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* m, β	*Vx* σ_p	Depth [m]	m_1	β_1	m_2	β_2	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 4a	0,20	0,10	0-0,8	110,00	1,10	110,00	1,00	-	147,00	17,20	
Clay 1				0,8-1,8	10,05	-0,13	83,60	0,53	58,30	-	15,69	
Clay 2				1,8-2,8	9,02	-0,50	76,10	0,22	13,03	-	14,78	
Clay 3				2,8-4,0	9,46	-0,41	NC	NC	NC	-	14,98	213
Clay 4				4,0-5,0	21,38	-0,16	132,17	0,29	9,52	-	16,80	
Clay 5				5,0-6,12	16,12	-0,14	186,65	0,21	21,62	-	18,38	
Silt/Morr.				6,12-16,4	110,00	0,33	NC	NC	NC	-	18,38	
Dry Crust	Analysis 4b	0,40	0,23	0-0,8	120,00	1,20	120,00	1,00	-	156,10	17,20	
Clay 1				0,8-1,8	10,96	-0,11	91,20	0,58	61,91	-	15,69	
Clay 2				1,8-2,8	9,84	-0,45	83,01	0,24	13,84	-	14,78	
Clay 3				2,8-4,0	10,32	-0,36	NC	NC	NC	-	14,98	185
Clay 4				4,0-5,0	23,32	-0,14	144,19	0,32	10,11	-	16,80	
Clay 5				5,0-6,12	17,59	-0,12	203,62	0,23	22,95	-	18,38	
Silt/Morr.				6,12-16,4	120,00	0,36	NC	NC	NC	-		
Dry Crust	Analysis 4c	0,70	0,35	0-0,8	135,00	1,35	135,00	1,00	-	164,50	17,20	
Clay 1				0,8-1,8	12,33	-0,09	102,60	0,65	65,24	-	15,69	
Clay 2				1,8-2,8	11,07	-0,36	93,39	0,27	14,58	-	14,78	
Clay 3				2,8-4,0	11,61	-0,29	NC	NC	NC	-	14,98	155
Clay 4				4,0-5,0	26,24	-0,12	162,21	0,36	10,65	-	16,80	
Clay 5				5,0-6,12	19,79	-0,10	229,08	0,26	24,19	-	18,38	
Silt/Morr.				6,12-16,4	135,00	0,41	NC	NC	NC	-	18,38	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Haarajoki case

Settlements - Compression Index Method
Best Estimates of soil parameters

Layer	Analysis (See: Table 4)	V_x Cc, Cs	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1a	0,0	0,0	0 - 2	1,32	0,27	0,06	-	49,40	17,17	
Clay 1				2 - 6	3,15	2,74	0,19	-	53,13	14,02	719
Clay 2				6 - 13	2,54	1,97	0,18	36,56	-	14,69	

Settlements - Compression Index Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table 4)	V_x Cc, Cs	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0.10	0.10	0 - 2	1.32	0.29	0.06	-	45.74	17.17	
Clay 1				2 - 6	3.15	3.21	0.20	-	50.04	14.02	884
Clay 2				6 - 13	2.54	2.07	0.19	34.55	-	14.69	
Dry Crust	Analysis 3b	0.24	0.23	0 - 2	1.32	0.32	0.07	-	40.99	17.17	
Clay 1				2 - 6	3.15	3.46	0.22	-	46.04	14.02	1142
Clay 2				6 - 13	2.54	2.23	0.20	31.93	-	14.69	
Dry Crust	Analysis 3c	0.37	0.35	0 - 2	1.32	0.34	0.07	-	36.61	17.17	
Clay 1				2 - 6	3.15	3.69	0.24	-	42.34	14.02	1420
Clay 2				6 - 13	2.54	2.37	0.22	29.52	-	14.69	

Settlements - Compression Index Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table	V_x Cc, Cs	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0.10	0.10	0 - 2	1.32	0.25	0.05	-	53.06	17.17	
Clay 1				2 - 6	3.15	2.86	0.18	-	56.21	14.02	578
Clay 2				6 - 13	2.54	1.86	0.17	38.57	-	14.69	
Dry Crust	Analysis 3b	0.24	0.23	0 - 2	1.32	0.22	0.05	-	57.81	17.17	
Clay 1				2 - 6	3.15	2.61	0.16	-	60.21	14.02	443
Clay 2				6 - 13	2.54	1.71	0.15	41.18	-	14.69	
Dry Crust	Analysis 3c	0.37	0.35	0 - 2	1.32	0.19	0.04	-	62.19	17.17	
Clay 1				2 - 6	3.15	2.38	0.14	-	63.91	14.02	306
Clay 2				6 - 13	2.54	1.57	0.14	43.59	-	14.69	

Settlements - Compression Index Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table 4)	V_x Cc, Cs	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0.10	0.10	0 - 2	1.32	0.28	0.06	-	46.93	17.17	
Clay 1				2 - 6	3.15	3.19	0.20	-	50.47	14.02	859
Clay 2				6 - 13	2.54	2.07	0.19	34.73	-	14.69	
Dry Crust	Analysis 5b	0.24	0.23	0 - 2	1.32	0.30	0.06	-	43.72	17.17	
Clay 1				2 - 6	3.15	3.40	0.21	-	47.02	14.02	1075
Clay 2				6 - 13	2.54	2.20	0.20	32.35	-	14.69	
Dry Crust	Analysis 5c	0.37	0.35	0 - 2	1.32	0.32	0.07	-	40.76	17.17	
Clay 1				2 - 6	3.15	3.60	0.22	-	43.83	14.02	1304
Clay 2				6 - 13	2.54	2.33	0.21	30.16	-	14.69	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Haarajoki case

Settlements - Compression Index Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table	*Vx* Cc, Cs	σp	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0.10	0.10	0 - 2	1.32	0.25	0.05	-	51.87	17.17	
Clay 1				2 - 6	3.15	2.88	0.18	-	55.78	14.02	597
Clay 2				6 - 13	2.54	1.87	0.17	38.38	-	14.69	
Dry Crust	Analysis 5b	0.24	0.23	0 - 2	1.32	0.24	0.05	-	55.08	17.17	
Clay 1				2 - 6	3.15	2.67	0.17	-	59.23	14.02	459
Clay 2				6 - 13	2.54	1.73	0.16	40.76	-	14.69	
Dry Crust	Analysis 5c	0.37	0.35	0 - 2	1.32	0.22	0.05	-	58.05	17.17	
Clay 1				2 - 6	3.15	2.47	0.15	-	62.42	14.02	352
Clay 2				6 - 13	2.54	1.60	0.15	42.95	-	14.69	

Kujala case

Settlements - Compression Index Method
Best Estimates of soil parameters

Layer	Analysis (See: Table 4)	*Vx* Cc, Cs	σp	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1a	0,0	0,0	0-2	-	-	-		-	19,20	
Silt 1				2 - 3,2	-	-	-	400,00	-	18,09	
Clay 1				3,2 - 3,9	1,32	0,53	0,10	80,00	-	16,08	
Clay 2				3,9 - 6,0	1,04	0,20	0,04	80,00	-	18,09	
Silt 2				6,0 - 6,9	-	-	-	NC	-	18,59	136
Clay 3				6,9 - 10	0,90	0,13	0,03	100,00	-	18,09	
Clay 4				10 - 12,6	1,82	0,61	0,09	100,00	-	17,09	
Silt 3				12,6 - 13,5	1,63	0,49	0,08	100,00	-	16,67	

Settlements - Compression Index Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* Cc, Cs	σp	Depth	e_o	C_C	C_S	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2	-	-	-	380,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,56	0,10	76,00	16,080	
Clay 2				3,9 - 6,0	1,04	0,21	0,05	76,00	18,090	
Silt 2				6,0 - 6,9	-	-	-	NC	18,593	147
Clay 3				6,9 - 10	0,90	0,14	0,04	95,00	18,090	
Clay 4				10 - 12,6	1,82	0,64	0,10	95,00	17,085	
Silt 3				12,6 - 13,5	1,63	0,51	0,08	95,00	16,672	
Dry Crust	Analysis 5b	0,24	0,23	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2	-	-	-	354,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,60	0,11	70,80	16,08	
Clay 2				3,9 - 6,0	1,04	0,22	0,05	70,80	18,09	
Silt 2				6,0 - 6,9	-	-	-	NC	18,59	167
Clay 3				6,9 - 10	0,90	0,15	0,04	88,50	18,09	
Clay 4				10 - 12,6	1,82	0,68	0,11	88,50	17,09	
Silt 3				12,6 - 13,5	1,63	0,54	0,08	88,50	16,67	
Dry Crust	Analysis 5c	0,37	0,35	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2	-	-	-	330,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,63	0,11	66,00	16,08	
Clay 2				3,9 - 6,0	1,04	0,23	0,05	66,00	18,09	
Silt 2				6,0 - 6,9	-	-	-	NC	18,59	184
Clay 3				6,9 - 10	0,90	0,16	0,04	82,50	18,09	
Clay 4				10 - 12,6	1,82	0,72	0,11	82,50	17,09	
Silt 3				12,6 - 13,5	1,63	0,58	0,09	82,50	16,67	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Kujala case

Settlements - Compression Index Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis	*Vx* Cc, Cs	σ_p	Depth	e_o	C_C	C_S	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2				420,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,51	0,09	84,00	16,080	
Clay 2				3,9 - 6,0	1,04	0,19	0,04	84,00	18,090	
Silt 2				6,0 - 6,9				NC	18,593	123
Clay 3				6,9 - 10	0,90	0,12	0,03	105,00	18,090	
Clay 4				10 - 12,6	1,82	0,58	0,09	105,00	17,085	
Silt 3				12,6 - 13,5	1,63	0,46	0,07	105,00	16,672	
Dry Crust	Analysis 5b	0,24	0,23	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2				446,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,47	0,08	89,20	16,08	
Clay 2				3,9 - 6,0	1,04	0,17	0,04	89,20	18,09	
Silt 2				6,0 - 6,9				NC	18,59	112
Clay 3				6,9 - 10	0,90	0,12	0,03	111,50	18,09	
Clay 4				10 - 12,6	1,82	0,54	0,08	111,50	17,09	
Silt 3				12,6 - 13,5	1,63	0,43	0,07	111,50	16,67	
Dry Crust	Analysis 5c	0,37	0,35	0-2	-	-	-	-	19,20	
Silt 1				2 - 3,2				470,00	18,09	
Clay 1				3,2 - 3,9	1,32	0,43	0,08	94,00	16,08	
Clay 2				3,9 - 6,0	1,04	0,16	0,04	94,00	18,09	
Silt 2				6,0 - 6,9				NC	18,59	100
Clay 3				6,9 - 10	0,90	0,11	0,03	117,50	18,09	
Clay 4				10 - 12,6	1,82	0,50	0,08	117,50	17,09	
Silt 3				12,6 - 13,5	1,63	0,40	0,06	117,50	16,67	

Murro case

Settlements - Compression Index Method
Best Estimates of soil parameters

Layer	Analysis (See: Table 4)	*Vx* Cc, Cs	σ_p	Depth [m]	e_o	C_C	C_S	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 1a	0,0	0,0	0-1,6	1,49	0,48	0,01	69,57	16,08	
Clay 1				1,6-3,0	1,73	0,59	0,02	NC	14,96	
Clay 2				3,0-7,5	2,40	1,07	0,03	NC	14,45	986
Clay 3				7,5-10	2,16	0,94	0,01	NC	15,43	
Clay 4				10,0-15,0	1,81	0,78	0,02	NC	15,41	
Clay 5				15-21,5	1,55	0,57	0,01	NC	16,37	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Murro case

Settlements - Compression Index Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table	V_x Cc, Cs	σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0,10	0,10	0-1,6	1,49	0,51	0,01	65,53		16,08	
Clay 1				1,6-3,0	1,73	0,65	NC	NC		14,96	
Clay 2				3,0-7,5	2,40	1,12	NC	NC		14,45	1055
Clay 3				7,5-10	2,16	1,04	NC	NC		15,43	
Clay 4				10,0-15,0	1,81	0,82	NC	NC		16,37	
Clay 5				15-21,5	1,55	0,61	NC	NC		16,37	
Dry Crust	Analysis 3b	0,24	0,23	0-1,6	1,49	0,55	0,01	60,29		16,08	
Clay 1				1,6-3,0	1,73	0,75	0,03	NC		14,96	
Clay 2				3,0-7,5	2,40	1,19	0,03	NC		14,45	1152
Clay 3				7,5-10	2,16	1,20	0,02	NC		15,43	
Clay 4				10,0-15,0	1,81	0,87	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,66	0,01	NC		16,37	
Dry Crust	Analysis 3c	0,37	0,35	0-1,6	1,49	0,58	0,01	55,44		16,08	
Clay 1				1,6-3,0	1,73	0,84	0,03	NC		14,96	
Clay 2				3,0-7,5	2,40	1,26	0,03	NC		14,45	1241
Clay 3				7,5-10	2,16	1,34	0,02	NC		15,43	
Clay 4				10,0-15,0	1,81	0,92	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,71	0,01	NC		16,37	

Settlements - Compression Index Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table	V_x Cc, Cs	σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0,10	0,10	0-1,6	1,49	0,46	0,01	73,60		16,08	
Clay 1				1,6-3,0	1,73	0,52	0,02	NC		14,96	
Clay 2				3,0-7,5	2,40	1,01	0,02	NC		14,45	917
Clay 3				7,5-10	2,16	0,83	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,74	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,53	0,01	NC		16,37	
Dry Crust	Analysis 3b	0,24	0,23	0-1,6	1,49	0,42	0,01	78,85		16,08	
Clay 1				1,6-3,0	1,73	0,42	0,02	NC		14,96	
Clay 2				3,0-7,5	2,40	0,94	0,02	NC		14,45	821
Clay 3				7,5-10	2,16	0,68	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,69	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,48	0,01	NC		16,37	
Dry Crust	Analysis 3c	0,37	0,35	0-1,6	1,49	0,38	0,01	83,69		16,08	
Clay 1				1,6-3,0	1,73	0,33	0,01	NC		14,96	
Clay 2				3,0-7,5	2,40	0,87	0,02	NC		14,45	731
Clay 3				7,5-10	2,16	0,53	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,64	0,01	NC		16,37	
Clay 5				15-21,5	1,55	0,43	0,01	NC		16,37	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Murro case

Settlements - Compression Index Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table	Vx Cc, Cs	Vx σp	Depth [m]	e_o	C_C	C_S	POP	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-1,6	1,49	0,51	0,01	66,09	16,08	
Clay 1				1,6-3,0	1,73	0,62	NC	NC	14,96	
Clay 2				3,0-7,5	2,40	1,12	NC	NC	14,45	1036
Clay 3				7,5-10	2,16	0,98	NC	NC	15,43	
Clay 4				10,0-15,0	1,81	0,82	NC	NC	16,37	
Clay 5				15-21,5	1,55	0,60	NC	NC	16,37	
Dry Crust	Analysis 5b	0,24	0,23	0-1,6	1,49	0,54	0,01	61,57	16,08	
Clay 1				1,6-3,0	1,73	0,66	NC	NC	14,96	
Clay 2				3,0-7,5	2,40	1,19	NC	NC	14,45	1104
Clay 3				7,5-10	2,16	1,05	NC	NC	15,43	
Clay 4				10,0-15,0	1,81	0,87	NC	NC	16,37	
Clay 5				15-21,5	1,55	0,64	NC	NC	16,37	
Dry Crust	Analysis 5c	0,37	0,35	0-1,6	1,49	0,57	0,01	57,39	16,08	
Clay 1				1,6-3,0	1,73	0,69	0,03	NC	14,96	
Clay 2				3,0-7,5	2,40	1,26	0,03	NC	14,45	1167
Clay 3				7,5-10	2,16	1,11	0,02	NC	15,43	
Clay 4				10,0-15,0	1,81	0,92	0,02	NC	16,37	
Clay 5				15-21,5	1,55	0,68	0,01	NC	16,37	

Settlements - Compression Index Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table	Vx Cc, Cs	Vx σp	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-1,6	1,49	0,46	0,01	66,09		16,08	
Clay 1				1,6-3,0	1,73	0,56	0,02	NC		14,96	
Clay 2				3,0-7,5	2,40	1,01	0,02	NC		14,45	937
Clay 3				7,5-10	2,16	0,89	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,74	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,54	0,01	NC		16,37	
Dry Crust	Analysis 5b	0,24	0,23	0-1,6	1,49	0,42	0,01	61,57		16,08	
Clay 1				1,6-3,0	1,73	0,52	0,02	NC		14,96	
Clay 2				3,0-7,5	2,40	0,94	0,02	NC		14,45	868
Clay 3				7,5-10	2,16	0,82	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,68	0,02	NC		16,37	
Clay 5				15-21,5	1,55	0,50	0,01	NC		16,37	
Dry Crust	Analysis 5c	0,37	0,35	0-1,6	1,49	0,39	0,01	57,39		16,08	
Clay 1				1,6-3,0	1,73	0,48	0,02	NC		14,96	
Clay 2				3,0-7,5	2,40	0,87	0,02	NC		14,45	804
Clay 3				7,5-10	2,16	0,76	0,01	NC		15,43	
Clay 4				10,0-15,0	1,81	0,63	0,01	NC		16,37	
Clay 5				15-21,5	1,55	0,46	0,01	NC		16,37	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Östersundom case

Settlements - Compression Index Method
Best Estimates of soil parameters

Layer	Analysis (See: Table 4)	V_x C_c, C_s	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 1a	0,0	0,0	0-0,8	-	-	-	-	140	17,32	
				0,8-1,8	2,062	0,89	0,12	55,53	-	15,69	
				1,8-2,8	2,694	1,92	0,11	12,41	-	14,78	
				2,8-4,0	2,619	1,77	0,14	NC	-	14,98	304
				4,0-5,0	1,577	0,58	0,07	9,06	-	16,80	
				5,0-6,12	1,035	0,37	0,04	20,59	-	18,38	
				6,12-16,4	-	-	-	NC	-	18,38	

Settlements - Compression Index Method
Lower bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table	V_x C_c, C_s	V_x σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0,10	0,10	0-0,8	-	-	-	-	133,00*	17,20	
Clay 1				0,8-1,8	2,06	0,93	0,12	52,92	-	15,85	
Clay 2				1,8-2,8	2,69	2,05	0,11	11,58	-	14,78	
Clay 3				2,8-4,0	2,62	1,89	0,15	NC	-	14,98	335
Clay 4				4,0-5,0	1,21	0,62	0,07	8,46	-	17,70	
Clay 5				5,0-6,12	1,03	0,39	0,04	19,06	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 3b	0,24	0,23	0-0,8	-	-	-	-	123,90	17,20	
Clay 1				0,8-1,8	2,06	0,99	0,13	49,52	-	15,85	
Clay 2				1,8-2,8	2,69	2,23	0,12	10,50	-	14,78	
Clay 3				2,8-4,0	2,62	2,06	0,17	NC	-	14,98	380
Clay 4				4,0-5,0	1,21	0,67	0,08	7,67	-	17,70	
Clay 5				5,0-6,12	1,03	0,42	0,04	17,08	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 3c	0,37	0,35	0-0,8	-	-	-	-	115,5	17,20	
Clay 1				0,8-1,8	2,06	1,05	0,14	46,39	-	15,85	
Clay 2				1,8-2,8	2,69	2,39	0,13	9,50	-	14,78	
Clay 3				2,8-4,0	2,62	2,21	0,18	NC	-	14,98	427
Clay 4				4,0-5,0	1,21	0,71	0,08	6,94	-	17,70	
Clay 5				5,0-6,12	1,03	0,46	0,05	15,25	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Östersundom

Settlements - Compression Index Method
Upper bound of characteristic values - October 2019 Formula

Layer	Analysis (See: Table	V_X C_c, C_s	V_X σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 3a	0,10	0,10	0-0,8	-	-	-	-	147,00	17,20	
Clay 1				0,8-1,8	2,06	0,85	0,11	58,14	-	15,85	
Clay 2				1,8-2,8	2,69	1,79	0,10	13,24	-	14,78	
Clay 3				2,8-4,0	2,62	1,65	0,13	NC	-	14,98	275
Clay 4				4,0-5,0	1,21	0,54	0,06	9,67	-	17,70	
Clay 5				5,0-6,12	1,03	0,34	0,03	22,11	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 3b	0,24	0,23	0-0,8	-	-	-	-	156,10	17,20	
Clay 1				0,8-1,8	2,06	0,79	0,10	61,53	-	15,85	
Clay 2				1,8-2,8	2,69	1,61	0,09	14,33	-	14,78	
Clay 3				2,8-4,0	2,62	1,49	0,12	NC	-	14,98	238
Clay 4				4,0-5,0	1,21	0,49	0,06	10,46	-	17,70	
Clay 5				5,0-6,12	1,03	0,31	0,03	24,09	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 3c	0,37	0,35	0-0,8	-	-	-	-	164,50	17,20	
Clay 1				0,8-1,8	2,06	0,74	0,10	64,66	-	15,85	
Clay 2				1,8-2,8	2,69	1,44	0,08	15,32	-	14,78	
Clay 3				2,8-4,0	2,62	1,33	0,11	NC	-	14,98	209
Clay 4				4,0-5,0	1,21	0,45	0,05	11,19	-	17,70	
Clay 5				5,0-6,12	1,03	0,27	0,03	25,92	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	

Settlements - Compression Index Method
Lower bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table	V_X C_c, C_s	V_X σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-0,8	-	-	-	-	133,00*	17,20	
Clay 1				0,8-1,8	2,062333	0,94	0,12	52,75	-	15,85	
Clay 2				1,8-2,8	2,694333	2,01	0,11	11,79	-	14,78	320
Clay 3				2,8-4,0	2,6185	1,86	0,15	NC	-	14,98	
Clay 4				4,0-5,0	1,2135	0,61	0,07	8,61	-	17,70	
Clay 5				5,0-6,12	1,0345	0,38	0,04	19,56	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 5b	0,24	0,23	0-0,8	-	-	-	-	123,90	17,20	
Clay 1				0,8-1,8	2,062333	1,00	0,13	49,14	-	15,85	
Clay 2				1,8-2,8	2,694333	2,15	0,12	10,99	-	14,78	
Clay 3				2,8-4,0	2,6185	1,99	0,16	NC	-	14,98	361
Clay 4				4,0-5,0	1,2135	0,65	0,07	8,02	-	17,70	
Clay 5				5,0-6,12	1,0345	0,41	0,04	18,22	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	
Dry Crust	Analysis 5c	0,37	0,35	0-0,8	-	-	-	-	115,5	17,20	
Clay 1				0,8-1,8	2,062333	1,06	0,14	45,81	-	15,85	
Clay 2				1,8-2,8	2,694333	2,27	0,13	10,24	-	14,78	
Clay 3				2,8-4,0	2,6185	2,10	0,17	NC	-	14,98	396
Clay 4				4,0-5,0	1,2135	0,69	0,08	7,48	-	17,70	
Clay 5				5,0-6,12	1,0345	0,43	0,04	16,98	-	18,38	
Silt/Morr.				6,12-16,4	-	-	-	-	-	18,38	

Appendix 6. Best estimates and characteristic values of soil parameters used in Compression Index method

Östersundom

Settlements - Compression Index Method
Upper bound of characteristic values - Schneider's Formula

Layer	Analysis (See: Table	V_X C_c, C_s	V_X σ_p	Depth [m]	e_o	C_C	C_S	POP	σ_p	γ'	Total settlement [mm]
Dry Crust	Analysis 5a	0,10	0,10	0-0,8	-	-	-	-	147,00	17,20	
Clay 1				0,8-1,8	2,06	0,85	0,11	58,30	-	15,85	
Clay 2				1,8-2,8	2,69	1,82	0,10	13,03	-	14,78	
Clay 3				2,8-4,0	2,62	1,68	0,14	NC	-	14,98	281
Clay 4				4,0-5,0	1,21	0,55	0,06	9,52	-	17,70	
Clay 5				5,0-6,12	1,03	0,35	0,04	21,62	-	18,38	
Silt/Morr.				6,12-16,4					-	18,38	
Dry Crust	Analysis 5b	0,24	0,23	0-0,8	-	-	-	-	156,10	17,20	
Clay 1				0,8-1,8	2,06	0,78	0,10	61,91	-	15,85	
Clay 2				1,8-2,8	2,69	1,69	0,09	13,84	-	14,78	
Clay 3				2,8-4,0	2,62	1,56	0,13	NC	-	14,98	254
Clay 4				4,0-5,0	1,21	0,51	0,06	10,11	-	17,70	
Clay 5				5,0-6,12	1,03	0,32	0,03	22,95	-	18,38	
Silt/Morr.				6,12-16,4					-	18,38	
Dry Crust	Analysis 5c	0,37	0,35	0-0,8	-	-	-	-	164,50	17,20	
Clay 1				0,8-1,8	2,06	0,73	0,09	65,24	-	15,85	
Clay 2				1,8-2,8	2,69	1,56	0,09	14,58	-	14,78	
Clay 3				2,8-4,0	2,62	1,45	0,12	NC	-	14,98	230
Clay 4				4,0-5,0	1,21	0,47	0,05	10,65	-	17,70	
Clay 5				5,0-6,12	1,03	0,30	0,03	24,19	-	18,38	
Silt/Morr.				6,12-16,4					-	18,38	